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Proceedings of the American Society of Civil Engineers

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WATERWAYS AND HARBORS DIVISION
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WATERFRONT STRUCTURE DESIGN FOR VARYING CONDITIONS[a]

William C. Stevens[1] and John S. Wilson[2]
(Proc. Paper 1639)

ABSTRACT

A review of the different designs of pier and wharf structures constructed by The Port of New York Authority during the past ten years is presented. The effects of site conditions and functional requirements on each of six major projects are discussed and the resulting construction described.

In many harbors the basic approach to the design of new waterfront structures may vary to a very minor extent except as new and improved techniques are developed or as basic functional requirements may change. However, many of the larger ports of the world possess sufficiently variable geologic and site conditions and a large enough disparity in berthing requirements as to present a constant challenge to the designer as he approaches each new pier or wharf problem.

Encompassing an area of 1500 square miles and containing 650 miles of developed or developable waterfront suitable for the berthing of ocean-going ships, the Port of New York presents a never ending series of problems in the design concept of new waterfront structures. Bed rock may out crop at wharf level or may be encountered at such great depths as to not warrant its consideration as a foundation solution or conversely as a problem. The rock may be overlain by boulders, sand, clay, silt in any number of combinations and of varying degrees of density. Subterranean structures such as subway, vehicular and utility tunnels criss-cross the harbor. Existing structures and facilities must be maintained, restored, extended as a part of the new construction in certain instances.

Note: Discussion open until October 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1639 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 3, May, 1958.

a. Presented at the Annual Convention of the American Society of Civil Engineers, New York, N. Y., October, 1957.

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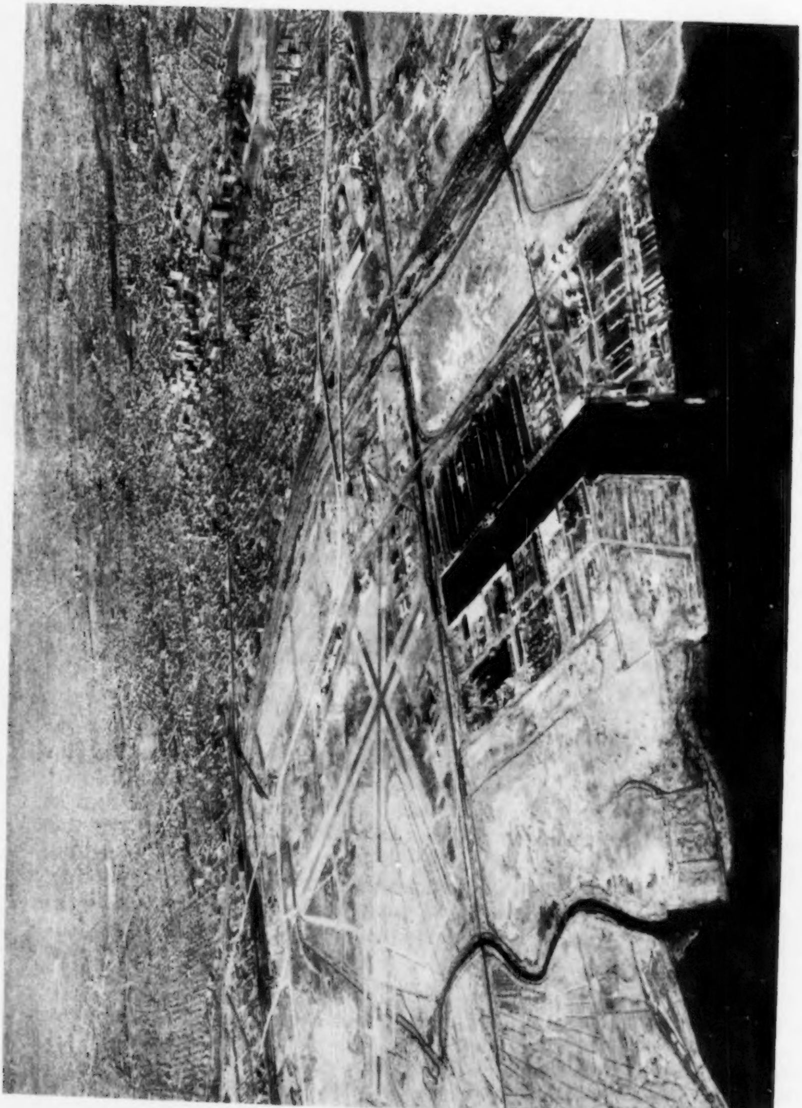

Fig. 1. Aerial View of Port Newark in Early Stages of Rehabilitation by the Port Authority

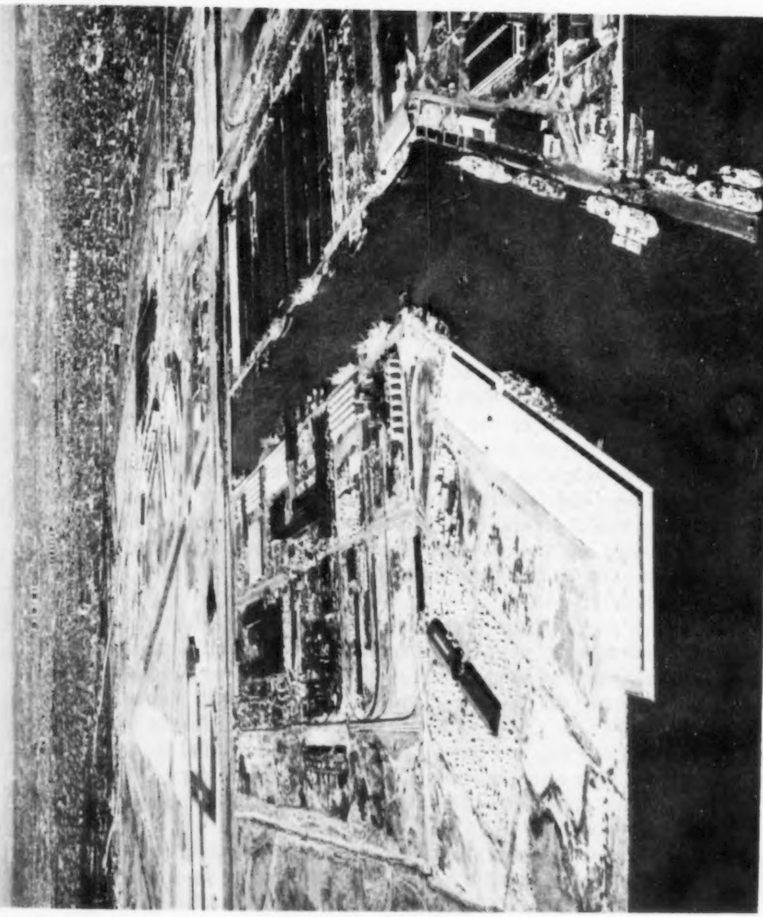

Fig. 2. Aerial View of Port Newark Waterman Project in the Foreground

This paper will attempt to deal with some of the specific problems encountered in the design of some of the waterfront structures constructed by The Port of New York Authority in its contribution towards the further development of the Port of New York.

Port Newark Wharfs

Since March, 1948, Port Newark (Fig. 1) operated by the Port Authority under a 50-year lease agreement with the City of Newark has been undergoing rehabilitation and development. To date over 28 million dollars has been invested in improvement and development of this port which last year handled over 2-1/2 million tons of cargo. Included in this investment are a dozen new transit sheds, warehouses and processing buildings comprising over one million square feet of enclosed, lighted, and in some cases, heated area, approximately one mile of new or completely rehabilitated wharfing space suitable for ocean-going vessels, many acres of open storage paved area and a planned improvement to the basic utilities and access areas. Additions of almost similar magnitude are now in the course of construction. Certain parts of that port's facilities were badly in need of repair and rehabilitation, but there also existed additional areas suitable for the development of new berthing sites as such needs might develop.

The first new area thus developed (Fig. 2) consisted of one open and three covered berths for lease to the Waterman Steamship Company. This involved the construction of 2200 feet of new wharf along the existing city channel and 550 feet around the corner or bay leg of this new L-shaped wharf. There existed an old timber bulkhead having a water depth of four feet along 2000 feet of the channel leg, and located forty-five feet inshore of the line of the face of the existing wharf structure to the west. Construction of the easterly 150 feet of the channel leg and the entire bay leg involved complete land reclamation in that area as the existing bay bulkhead was 100 feet inshore of the proposed new bulkhead line.

Rock in this area varies from 60 to 100 feet below mean low water and is overlain by clay and meadow mat. Preliminary designs and estimates were prepared on the basis of low bearing value (15 ton piles) versus higher bearing value piles (120 tons) driven to rock based on a super-imposed live load of 600 pounds per sq. ft. or Coopers E-50, plus the lateral thrust due to ship mooring (assumed at one kip per foot of wharf) and the lateral thrust of the upland area behind with a berthing depth at 9 feet from the stringpiece of thirty five feet. These studies resulted in the adoption of the higher bearing value piles driven to rock.

At the time, two basic types of structures were analyzed in various combinations, the high level type platform and the low level relieving type platform (Fig. 3). The latter provides greater flexibility for utilities and variations in future possible location of live loading requirements such as gantries and railroad trackage plus providing the necessary super-imposed dead load to take the vertical reaction of the batter pile opposing the horizontal thrust of earth pressure and mooring loads. Based upon prevailing costs at the time, the design analyses indicated that the low level relieving type platform was the most feasible. In order adequately to sustain the lateral loading, a forty-five foot width of platform was determined to be the required width.

As an illustration of the flexibility of the design finally adopted, it is interesting to note that following a change in operators at this facility it became

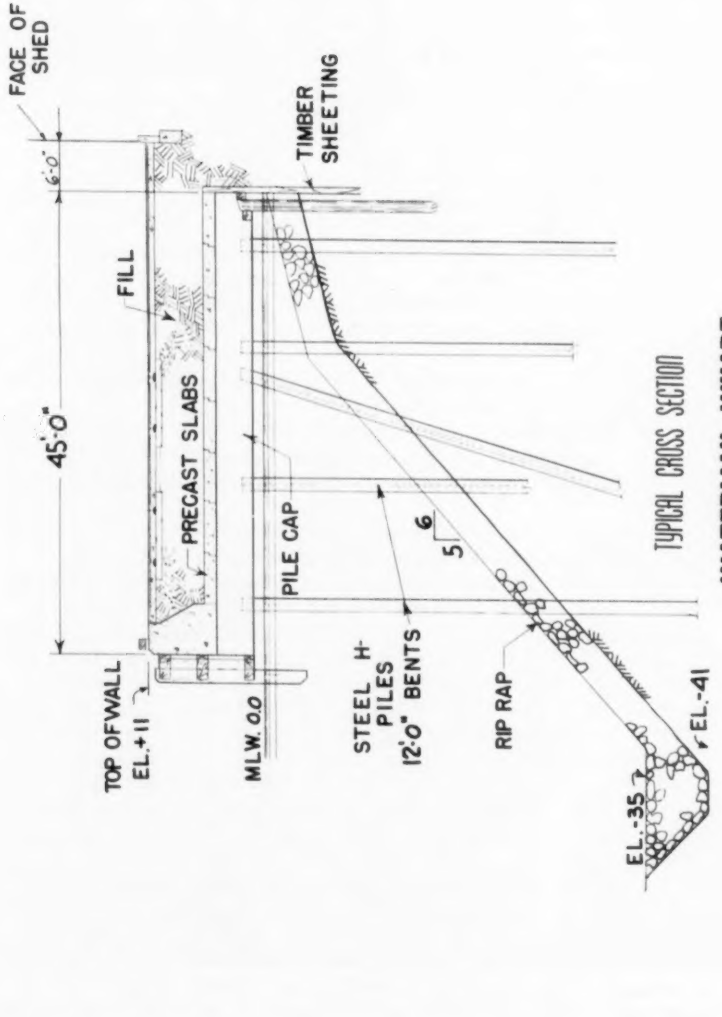

Fig. 3. Waterman Wharf Typical Cross Section

desirable to install a gantry crane on a portion of this wharf. The original design did not, of course, contemplate such a type of loading. The lessee had feared that it would be necessary to make radical and costly foundation modifications involving the addition of new bearing piles. On their request the problem resulting from the changes involved as a result of this loading was analyzed. By making relatively simple modifications above the lower deck, involving the addition of a deep crane rail beam it was possible to accommodate this change in the basic design criteria originally established without the addition of new piling or any basic modification in the original design.

In order to obtain and protect the required thirty-five feet of depth nine feet from the stringpiece, a rip rap protected slope from that depth with a pocketed bottom of slope and a slope of 1-1/4 to 1 was established, the rip rap varying in thickness from four feet at the base to three feet along the slope.

Conventional reinforced concrete pile bents twelve feet on centers with piles spaced approximately eleven feet on centers in each bent formed the supporting base for the structure with one batter pile per bent. In order to establish the necessary loading requirements fourteen BP 102# piles were selected having a bearing capacity of 120 tons per pile. This designed bearing capacity was verified by pile load tests before completion of the design. The lower deck, six feet below finished grade, was designed on the basis of pre-cast reinforced concrete slabs 12" in thickness with conventional steel reinforcing.

Key and mortar joints were provided in the slabs of sufficient size (three inches) to permit for normal tolerances.

In order to insure adequate life expectancy from the steel bearing piles, considerable study was made of prevailing ground and water conditions to determine the probable corrosive action which might take place on this ferrous material.

For many years ships' hulls have been protected against corrosion by establishing and maintaining a controlled electrical current flow from sacrificial anodes to the hull of the ship which acts as a cathode. A similar system of cathodic protection was developed for the wharf piling consisting of an interconnection of all piling at the cap level and the introduction of anodes for current supply buried in the rip rap slope. The necessary rectifiers for voltage and current control together with the required alarm and recording devices were installed in small sections of the 270,000 square feet of transit shed area constructed as part of this development. Test piling has been installed to measure the relative corrosive effect on protected versus unprotected piling. The results thus far have indicated that approximately ten times greater life can be expected from the protected piling thus installed over unprotected piling. The additional initial cost of this cathodic system, plus the capitalized operating cost seems to be justly warranted.

Because of the rip rap protected slope below the platform and extending out toward and beyond the stringpiece, it was not feasible to adopt a conventional piling type of fendering system because of material and maintenance re-installation problems through the rock protected slope and fendering and therefore, a suspended type of system consisting of a lattice work of 12" by 12" thick hardwood sections spaced at approximately 6' on centers vertically and 6' on centers horizontally was developed. The vertical rubbing pieces extended from the top of the stringpiece to 2' below mean low water and were spaced at 6' intervals. To date the maintenance of this system has proved to

Fig. 4. View of Completed Waterman Development

be quite economical. Further, the additional initial cost was minimized because of the ability to preassemble large portions of the entire system which was anchored to the outer concrete wall of the wharf by screwed bolt inserts.

The structure of the transit sheds (Fig. 4) comprising the covered berth area behind the wharf were constructed on pile foundations and were of rigid frame steel design. Floors of these buildings are constructed of flexible pavement founded on ground preconsolidated by means of the now conventional sand drain and surcharge method.

The outer skin of the buildings consists of asbestos protected metal from the floor line up. From the point of view of maintenance and operation it would appear more desirable to form the lower section of the outer walls of concrete or concrete block so as to reduce the costs of repair due to damaging of the lower portion by cargo handling equipment. However, in this area, the foundation conditions were sufficiently critical as to not warrant the additional initial cost of the foundations so involved by the superimposition of this additional dead load.

In order to reduce power consumption for lighting during daylight hours, skylights were installed in the roof to an extent of approximately two per cent of the roof area. Fluorescent lighting was installed having an average light intensity of two foot candles.

In 1955 the need for additional berthing and shedded area developed, and it became necessary to extend the bay leg of the wharf constructed previously for accommodation of the operations of the Norton, Lilly & Co. Steamship Line (Fig. 5). Their requirements were for four berths or 2400 feet of wharf and the construction of 360,000 square feet of transit sheds immediately behind.

This bay leg extension of the previously completed wharf to the north was approximately 200 feet outshore of the then existing shore line. The existing bottom and shore consisted generally of silt and spoilage from previous dredging operations in the silty channel of the bay. In order to provide the necessary new ground for the apron side of the new buildings and the buildings themselves, it was necessary to place 330,000 cubic yards of fill between the newly established bulkhead and the existing shore line. Further, because of the existing depth of water in the berthing and maneuvering area and because of the unsuitability of the underlying material for other purposes it was necessary to dispose of 1,350,000 cubic yards of such dredged material at sea.

General subsoil conditions follow the trend found for the previous development for Waterman to the north with rock at about elevation 80 feet below mean low water. General functional design requirements followed those previously established for the first development with the principal addition of a burtoning system on the sheds required by the prospective tenant.

Since the construction of the area for Waterman, sufficient advances had been made in the prestressed concrete field to warrant study of possible alternatives along this line. Further, because of the general steel market condition at the time, it was considered desirable to study the possible use of prestressed concrete piles in lieu of or as an alternate to the steel H pile used previously.

Again, alternate studies and preliminary designs and estimates were prepared for various types of wharf structures. Again, the low level relieving type platform proved to be the most promising and the final design of the structure proceeded along these general lines.

The final design developed (Fig. 6) produced a precast, prestressed

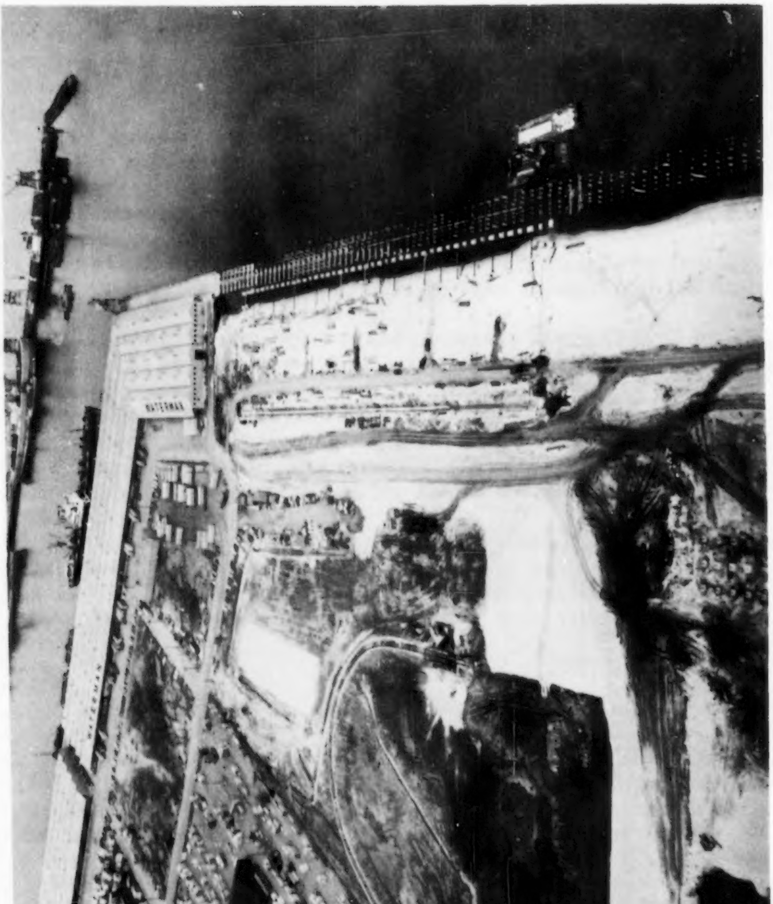

Fig. 5. View During Construction of Norton Lilly Development

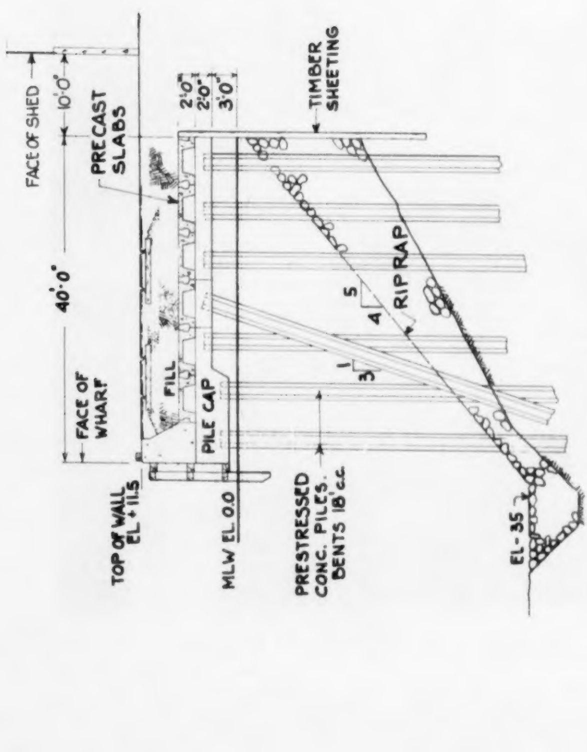

Fig. 6. Norton Lilly Wharf Typical Cross Section

bearing pile of octagonal shape with a diameter of 2' -0" and with a cylindrical void in the center ten inches in diameter. The piles have 14 3/8" steel strands stressed to fourteen kips each.

These piles were cast in sixty-foot lengths and driven to a bearing capacity of eighty tons per pile. Pile bents consisted of six vertical and two batter piles and the bents were spaced 18' on centers.

Conventional reinforced concrete caps three feet wide and 6 1/2 feet deep were used. The lower deck was designed using a precast, prestressed channel shaped concrete slab 16' long by 5' 10" wide. The flanges of the channel are 2' deep by 1' wide with a 6" web. The flanges contain 18 3/8" steel strands each, stressed to fourteen kips each and cut flush with the end of the slab.

Other general construction details followed the pattern previously established for the abutting wharf construction to the north except that with the use of concrete for the piling the need for a corrosive preventative was now eliminated and thus that cost due to operation of the cathodic system was also cancelled out.

The introduction of the center sonotube void in the pile facilitated jetting of the pile to within a few feet of final elevation in addition to the weight saving thus affected.

The building construction forming the transit sheds, (Fig. 7) now being completed as part of this development, generally follow the pattern developed for the previously constructed buildings forming part of the Waterman area, with some specific variations. The addition of facilities to the building structure to accommodate cargo burtoning has been previously mentioned. The lateral thrust of the burtoning system required modifications not only to the general building framing, but also to the foundation design, resulting in considerable additional initial cost.

Improvements in aluminum for siding and roofing, its lower initial cost than asbestos protected metal and the attendant dead weight reduction led to its use for siding and roofing. Skylight construction was simplified by the use of corrugated plastic at slight additional cost over normal roof construction.

Brooklyn New Pier 11

The first step in reconstruction of the 2-mile stretch of the Brooklyn Piers (Fig. 8) purchased by the Port Authority in 1956 involved the construction of New Pier 11. This will be a 2,00 foot long, 3 berth Marginal Wharf type situated in the old Atlantic Basin area and located 270 feet outshore of the old existing bulkhead.

Insufficient upland area existed between the original bulkhead line and the city street to permit the construction of necessary shedded area and provisions for adequate truck space on the landward side of the shed. It therefore became necessary to reclaim a 270-foot strip of land for these purposes.

The existing soil conditions in this area indicated an overlayer of approximately five feet of river silt over a sand strata of from ten to twenty feet. Below this a varved clay of 60 to 80 feet in thickness extended to the boulders and gravel overlying the bed rock.

The berthing depth at wharfside was established at 35 feet and live loading requirements along the 2000-foot apron was set at 500 pounds per square foot.

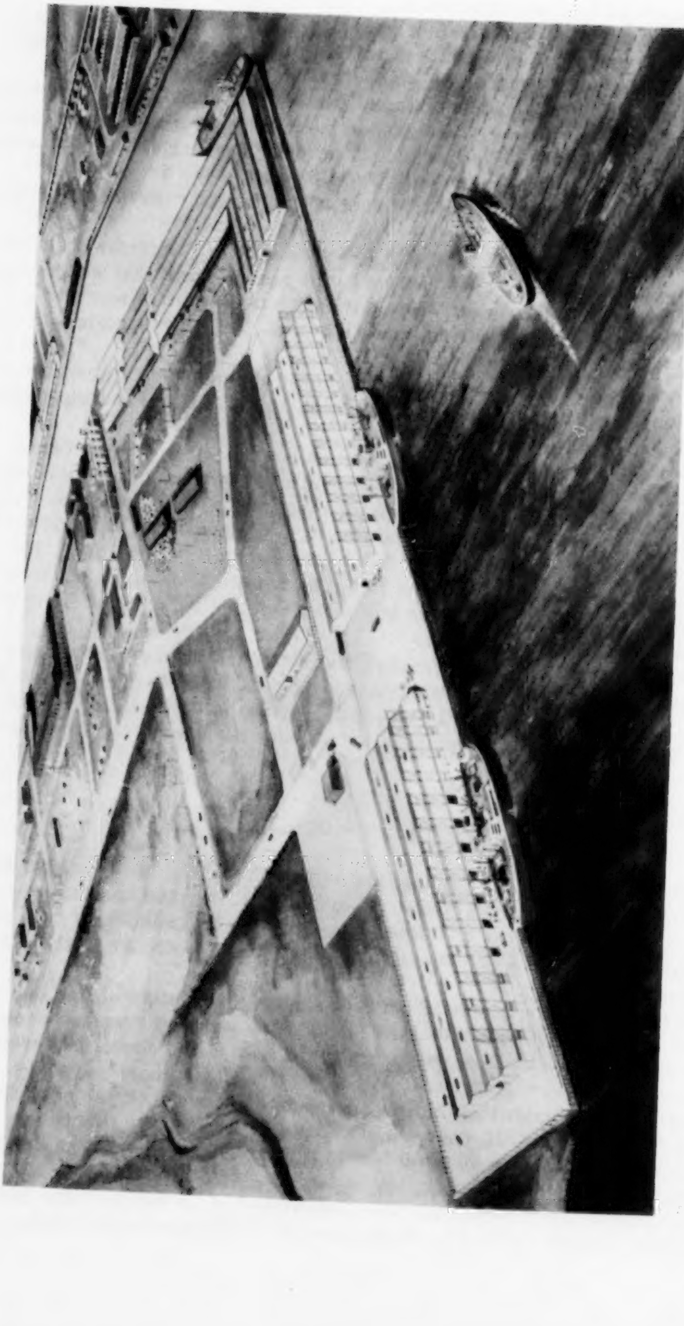

Fig. 7. Artist's Drawing of Completed Norton-Lilly Development

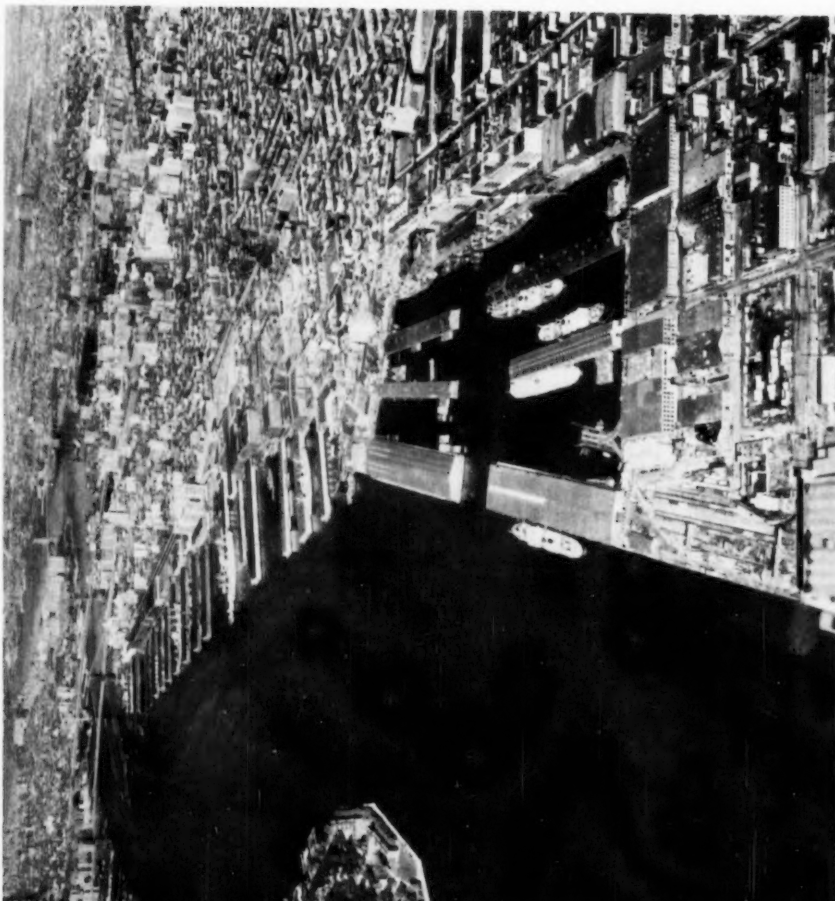

Fig. 8. Brooklyn - P. A. Piers at Time of Acquisition by the Port Authority - Atlantic Basin in the Foreground

With these factors in mind, various alternative designs were studied, and preliminary designs and comparative cost estimates prepared for the following types of basic wharf construction:

1. High level platform on timber piles (20 ton).
2. Low level platform on timber piles (20 ton).
3. Bulkhead with relieving platform on landward side.
4. Cofferdam construction
5. Soldier piles with steel sheeting placed between in tension.
6. Soldier piles with precast concrete slabs between.
7. Flexible steel sheet pile bulkhead.

After completing the foregoing analysis the flexible steel sheet pile bulkhead (Fig. 9) proved to be the most feasible provided some practical means were developed to provide a low water tie to the anchor sheeting.

It, of course, was necessary to locate the anchor sheeting consisting of a continuous wall of ZP-27 sheets, 18 feet in depth, far enough behind the bulkhead to be beyond the critical slope line as established by soil studies. This point was established at eighty four feet inshore of the main bulkhead line.

The problem then involved was to develop a practical means of installation of the anchor whaler for the tie back connection at the main bulkhead line at a sufficiently low elevation so as to minimize to as great a degree as possible, the moments developed in the bulkhead wall. Naturally, the use of divers, dewatering or other such methods would necessarily adversely affect the cost. Finally, a method was developed by slotting, or omitting the upper portion of sections of the sheet steel pile bulkhead at nine-foot intervals so as to permit dropping the tie rod through such slots.

In the final design as installed, the walers, consisting of twelve WF fifty sections and each eighteen feet in length, together with two forty-two foot lengths of 3 5/8 inch diameter tie rod are dropped through slots in the sheeting.

A complete outshore waler, together with two tie rods (spaced at 9'-0" c.c.) is then dropped through the slot formed by omission of the upper section of one sheet. The inshore end of the tie rod is then connected to and tightened against the anchor waler. Thus, with the lower end of the sheeting anchored in the same strata and an intermediate support from the anchor sheeting obtained five feet below mean low water, a partially fixed-end cantilever is effected with an intermediate support twenty-five feet from the fixed end and a free span above the intermediate support of seventeen feet. This design permitted the use of ZP38 sheeting for this designed depth of thirty-five feet of water with a twelve foot elevation of the wharf apron.

A continuous reinforced concrete cap piece, with an embedment of the sheeting of eighteen inches into the concrete, forms the top finish of the bulkhead and provides the base for the mooring devices and the hung fendering system. In order to prolong the life of the steel sheet piles of the main and anchor bulkheads, the tie rods and the walers, a cathodic protection system was installed.

The 270,000 square feet of transit shed (Fig. 10) located behind the Wharf will be of aluminum sheeting and roofing supported by light weight Fink trusses with a fifty-foot span with the bents forty feet on centers. Columns will be supported on spread footings and the floors of the shed of asphaltic concrete constructed on the pre-consolidated base of the imported fill forming the reclaimed land in the area. In order to minimize future differential

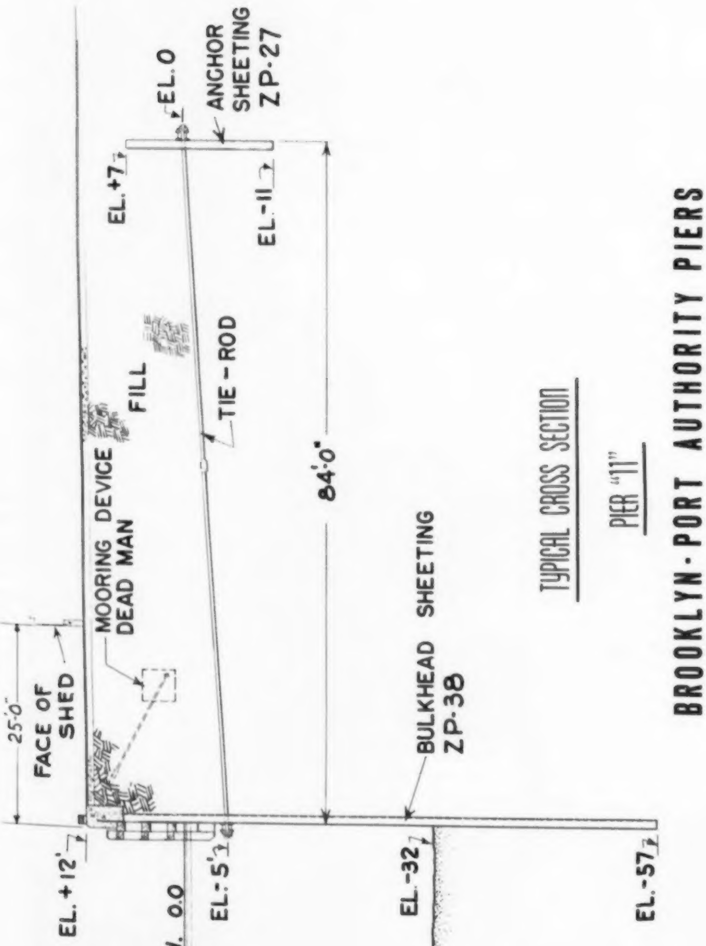

Fig. 9. Brooklyn - P.A. Piers - Pier 11 - Typical Cross Section

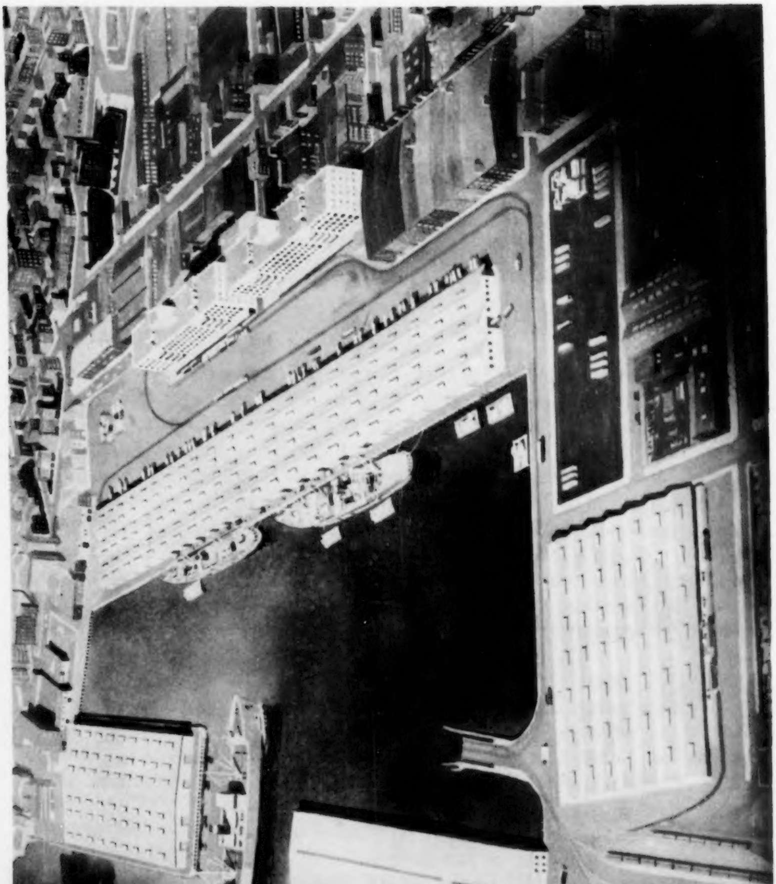

Fig. 10. Artist's Drawing of Completed Pier 11 at Right and Atlantic Basin Area

settlement, the existing underlying silt in the area to be reclaimed was to be carefully swept during the initial dredging operations necessary to form a good base for the selected fill to be brought in for the development.

Hoboken Finger Piers

The first major finger piers constructed by the Port Authority were Piers "A" and "C" at Hoboken, New Jersey (Fig. 11).

The site was leased to the Port Authority under a joint agreement with the United States Maritime Administration and the City of Hoboken in 1952. At that time, the leased area consisted of paved upland, headhouses, and Piers 1, 2 and 3 of an original six which were built around the turn of the century. The substructures of these piers, which were still in use everyday, were of the typical timber construction prevalent at the time of construction. Condition surveys established that the timbers above the mid-tide range on Piers 1 and 2 were in a state of advanced decay, while those on Pier 3 were somewhat less deteriorated.

For the design of any new pier structures, certain basic criteria were established by the Port Authority. The pier shed would be a single-story structure, and provide 90,000 square feet of covered storage area for each of the two berths. The aprons alongside the shed would be 25' wide, and the end apron 20' wide. The shed would have a column spacing of approximately 57' x 64' for ease of truck maneuverability. Provision of railroad trackage on the piers was a requirement of the Maritime Administration, which body retains the right to the use of these piers in the event of a national emergency. Berthing depth alongside would be 35' minimum. Substructure construction was to be fireproof.

The deck design live loads were established as 600 lbs. per square foot or the H20-S16 truck loading for the storage areas, and Cooper's E-50 railroad loading for the trackage. Lateral loading on the deck was to be that imparted by a berthing vessel of 10,000 short tons moving at one foot per second, with forty per cent of that load resisted by the structure.

With the establishment of the physical condition of the existing structures and of the basic criteria for new construction, economic studies for the modernization of the entire area were prepared by the firm of Parsons, Brinckerhoff, Hall & MacDonald and reviewed by the Port Authority. These initial studies contemplated construction of three new piers as well as complete renovation of the headhouses and repaving of the upland. The design of the first new pier, then designated as Pier "C," was begun in 1953.

Subsequent to the original improvement studies, negotiations with the tenant, American Export Lines, led to a revision to the master plan for redevelopment of the area. Pier 3 could be used by the tenant in its present form, with slight modification, if it could be economically restored to operating condition. Studies of the pier indicated that it was possible to do this. As a result, the rehabilitation of that structure began in 1955 concurrently with the construction of the second new pier, then designated Pier "A" (Fig. 12).

To satisfy the area requirements, each new pier would be approximately 323' wide and extend approximately 700' out to the U. S. Pierhead Line.

Extensive borings taken at the site revealed that the entire area of the new piers is underlain by from 50 to 130 feet of organic silts, all very soft and incapable of carrying even lightly loaded piles without undue settlement.

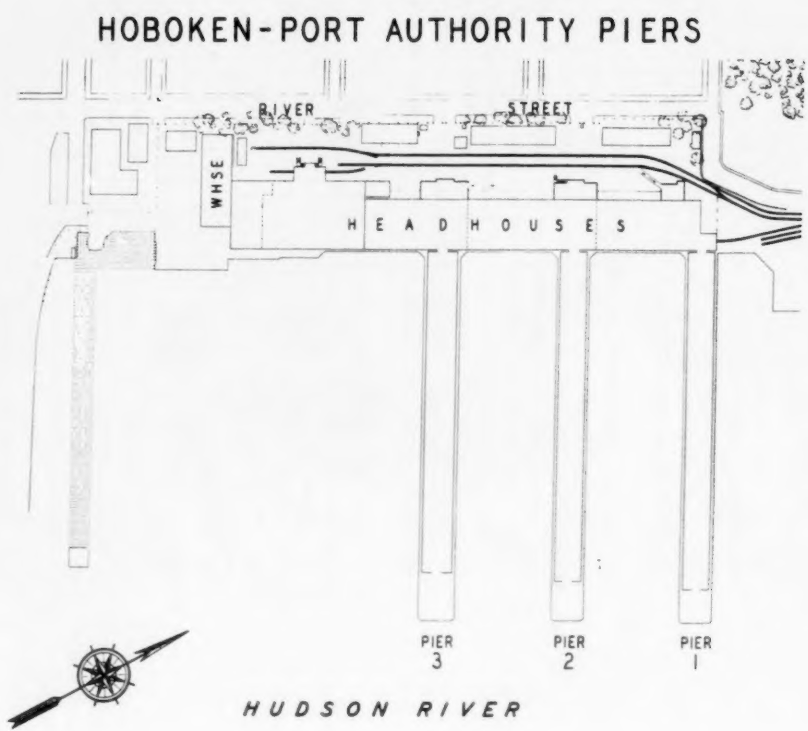

Fig. 11. Hoboken - P.A. Piers - Site Plan at Time of Acquisition by the Port Authority. Piles of Old Pier 6, Previously Destroyed by Fire, Are at the Far Left.

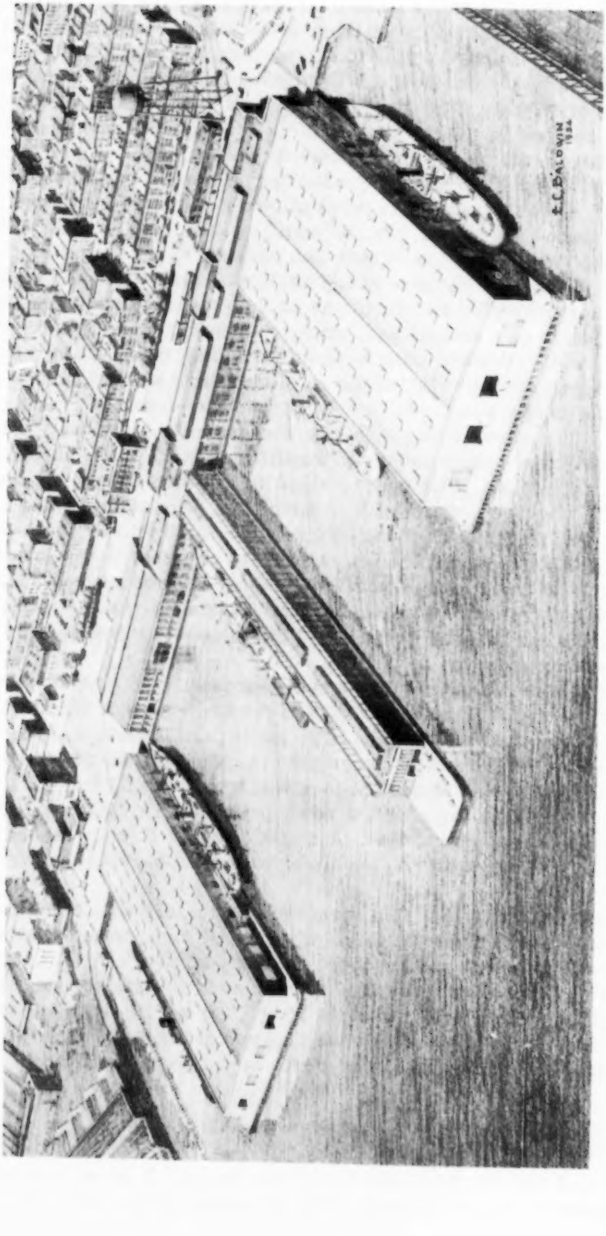

Fig. 12. Hoboken - P. A. Piers - Artist's Drawing of Completed Development

Along the bulkhead, a sand layer of variable thickness lies between the silt and bedrock. This sand layer terminates approximately half way out to the Pierhead Line. The old Piers 1, 2 and 3, supported on timber piles, had settled as much as two feet at the outshore end although the piles were designed to carry only fifteen tons. Most of those piles were over 80 feet long and were provided with timber lagging to increase their capacity. In view of these conditions, it was decided to utilize steel H-piles, driven to rock, with a design load of 90 tons. The pile lengths would vary from 100 feet at the inshore end to 150 feet at the outshore end of the pier. The use of precast concrete piles was precluded because of the excessive length.

As a result of extensive corrosion studies on similar structures in the Hudson River, it was decided that these piles should be protected by a cathodic system to prolong their useful life. In addition, the piles are coated with a bitumastic paint above the Low Water line because of the absence of other protection in that area. Because of the expected loss of metal, the choice of pile size is determined by that factor as well as by load requirements.

With the type of pile established, attention was then turned to the deck construction. As a result of their economic studies, the engineers found that two different deck designs offered the most economic solution to the problem (Fig. 13). One design would use cast-in-place, post-tensioned concrete pile caps on 21' - 6" centers with precast, pretensioned concrete deck slabs placed upon the caps. Each deck slab would be 3' x 20' x 1' thick and would be set on a 1" grout bed. The precast deck units would be knit together with a series of cast-in-place concrete filler strips and edge beams. The second design would call for cast-in-place concrete pile caps with conventional reinforcing, again at 21' - 6" centers, with precast concrete beams placed in pockets in the pile caps. These beams would not be prestressed. A 12" cast-in-place deck slab with conventional rod reinforcement would complete the deck. A wearing surface of two inches of bituminous concrete would be applied to either surface.

Because of the developing interest in prestressed concrete construction and a desire to develop a more economical design, it was decided that the two designs should be bid competitively. The successful bidder, J. Rich Steers, chose the prestressed concrete design, and the pier was so constructed.

The design of Pier "A" is almost identical with that of Pier "C." The only modification of note was the use of a wider precast deck slab unit to reduce the number of pieces to be handled. It might be noted that the tenant has filled the railroad track wells on each pier to permit unrestricted truck operations.

In each case, the pier shed is a steel truss structure with a flat roof and asbestos-protected metal roofing and siding. An 8' concrete cheek wall has been provided between the rolling steel shutter doors.

The fender system used on each of these piers consists of a series of Greenheart piles spaced approximately 7' - 6" on center, with a double row of walers and chocks. Mooring bitts are spaced at 60' centers, with net rings at 20' centers between the bitts. No cleats have been provided.

Brooklyn New Pier 2

The second of the new Brooklyn Piers will be Pier 2, to be located approximately one-quarter of a mile south of the Brooklyn Bridge (Fig. 14). This

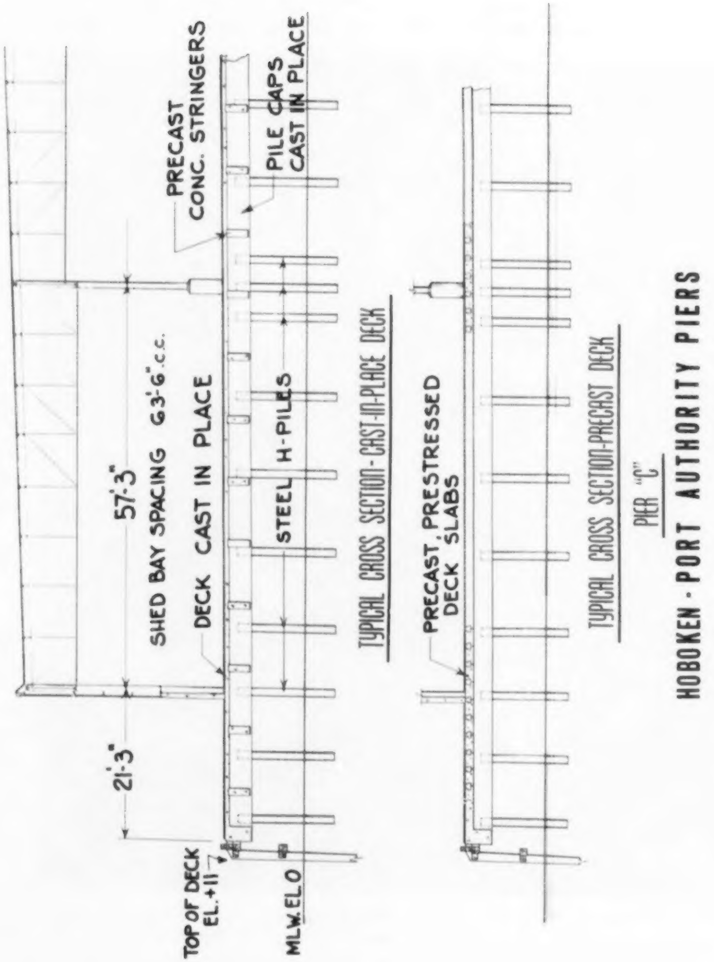

Fig. 13. Hoboken P. A. Piers - Pier "C" - Alternate Deck Designs

Fig. 14. Brooklyn - P. A. Piers - Aerial View Showing Piers now under Construction

pier will occupy the sites of existing Piers 8, 9, 9 1/2 and 10, all of which are of typical timber construction and no longer serviceable. The new pier will be situated such that its southern quarter will span the IRT Subway Tunnels at the foot of Clark Street. That site is now occupied by Pier 9 1/2, which is a protective structure rather than a functioning pier.

As before, the basic criteria for the design were first established. The pier is again to provide 90,000 square feet of covered area for each of the two berths, with 25' side aprons and a 20' end apron. The shed will provide a column spacing of approximately 40' x 58'. In this case, no railroad trackage will be provided. Berthing depth alongside is to be 32' minimum. As before, substructure construction is to be fireproof.

The deck design live load is 500 lbs. per square foot or the B20-S16 truck loading, and the lateral load is the same as for the Hoboken Piers (that imparted by a vessel of 10,000 tons).

Prior to the preparation of economic studies by Port Authority engineers, a series of borings were made at the construction site. These borings disclosed a layer of approximately twenty feet of soft organic silt, a dense sand layer from fifteen to twenty feet thick, and a silty clay varying in thickness from 30 feet at the bulkhead line to 70 feet at the Pierhead line. Beneath this clay layer lies bed rock, which is from 90 to 130 feet below Mean Low Water.

It was apparent from a study of these subsurface conditions that two types of piles could be used; a heavily-loaded steel pile driven to rock or a lightly-loaded timber pile supported in the sand layer. Precast concrete piles were again precluded by excessive length.

However, studies could not be finalized until the conditions for construction over the subway tunnels were resolved with The New York City Transit Authority. The subway tunnels in this case are two 17'-6" outside-diameter tubes, separated a clear distance of 9'-6", and varying in depth from 40' to 60' below Mean Low Water. In order to protect the tunnels, the Transit Authority required that any displacement piles must be driven a minimum of 6' from the face of the tunnels. This requirement immediately rules out the use of all but open-end pipe piles or steel H-piles between the tunnels. After an analysis of the spand and loads involved, it was determined that three rows of steel piles, designed for 100 tons each and either steel H-sections or concrete-filled pipe, would be used between and alongside of the tunnels.

With the basic conditions established, the economic studies were completed. It became evident that two different designs could be used, with but little apparent difference in cost (Fig. 15). The first design would be similar in most respects to the Hoboken Piers, using steel piles driven to rock for the entire substructure. Cast-in-place concrete pile caps, conventionally reinforced and spaced 20' apart, would support precast, pretensioned concrete deck slabs. The deck would then be knit together with the same cast-in-place fillers and edge beams. The construction over the subway tunnels would be a cast-in-place concrete slab, without pile caps. The entire deck area in this design would be covered with a 2" bituminous concrete wearing surface.

The second design, which could be utilized away from the subway tunnels, would use timber piles driven to twenty-ton resistance in bents spaced 10' apart, with a cast-in-place concrete slab without pile caps. Pile fireproofing would be accomplished either by a precast, pretensioned concrete extension driven onto the pile butts, or by pouring concrete into a precast concrete pipe form. Either type of fireproofing would extend from the mid-tide line to the pier deck. As with the first design, the construction over the subway tunnels

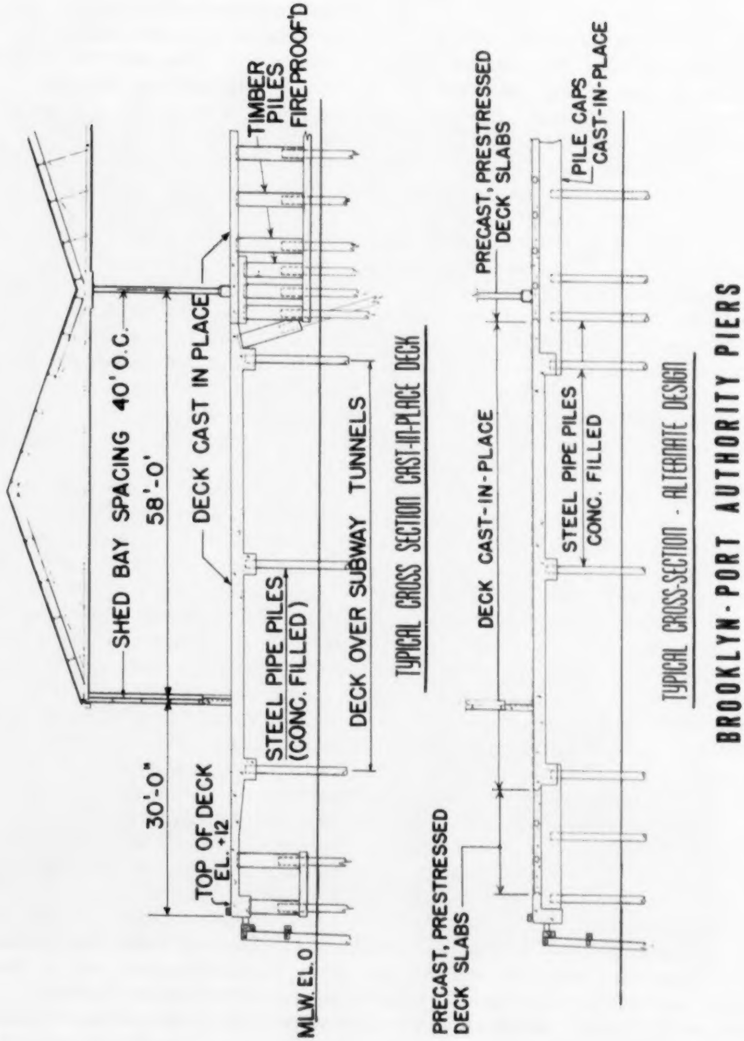

Fig. 15. Brooklyn - P. A. Piers - Pier 2 - Alternate Deck Designs

would be a cast-in-place concrete slab supported on steel piles. Because of the different types of piles involved in this design, the deck would be suitably jointed to allow for the expected differential settlement of the piles under load.

As before, cathodic protection and bitumastic paint would be required on any steel piles to be driven. The choice of size of the steel piles is again predicated upon the expected rate of corrosion in this area as well as by load requirements.

Concurrently with the design studies, load tests on timber and concrete-filled pipe piles were conducted to verify the design assumptions. In addition, the possibility of incorporating the existing timber piles of old Pier 9 into the new structure was considered. These untreated piles, while many years old, appeared to be sound below the mid-tide line. Load tests were conducted on representative piles, and certain others pulled for inspection. Such inspection revealed a continuing bacterial attack on the timber which had penetrated over one inch into the piles, making their reuse impossible.

After reviewing the results of the studies, it was again determined that the two deck designs should be bid competitively, with the optional types of fireproofing for timber piles and the use of either steel H-piles or concrete-filled pipe piles. In this manner, it was expected that maximum economy, compatible with good design, would be attained.

The successful bidder, Charles F. Vachris, has elected to use the timber pile design with the concrete jacket type of fireproofing in the areas away from the subway tunnels, and concrete-filled pipe piles in that area. He has been given permission to change the deck design over the subway area to expedite completion of the contract. The revised design will employ a series of precast, pretensioned beams seated upon a cast-in-place concrete capital atop each steel pile. Cast-in-place diaphragms between the beams and a cast-in-place deck slab will complete this construction. At this writing, pile driving operations are under way at the site.

The pier shed, which will be erected under a subsequent contract, will be a steel truss structure with aluminum roofing and siding. A concrete cheek wall 4' high will be provided between the rolling steel cargo doors.

The fender system will consist of a series of creosoted oak piles spaced 7'-6" on center, with two rows of creosoted oak walers and chocks. In addition, cylindrical rubber blocks, 10" in diameter and one foot long, will be placed behind each pile between the upper waler and the concrete edge beam. The employment of these rubber blocks is expected to prolong the life of the fender system as well as improve berthing conditions and reduce the lateral load on the pier deck. This is the first of the Port Authority piers to employ this type of system, which is expected to become standard on all future piers where conditions permit.

Brooklyn New Pier 1

The third of the new Brooklyn-Port Authority Piers will be Pier No. 1. Pier No. 1 is located directly south of the Brooklyn Bridge and will occupy the sites of existing Piers 3, 4, 5 and 6 (Fig. 16). Like Pier 2, it crosses subway tunnels, in this instance the Independent subway tunnels under Pier 6.

Unlike the piers previously described, the size and shape of Pier 1 is controlled largely by the location of the existing bulkhead and of the United States

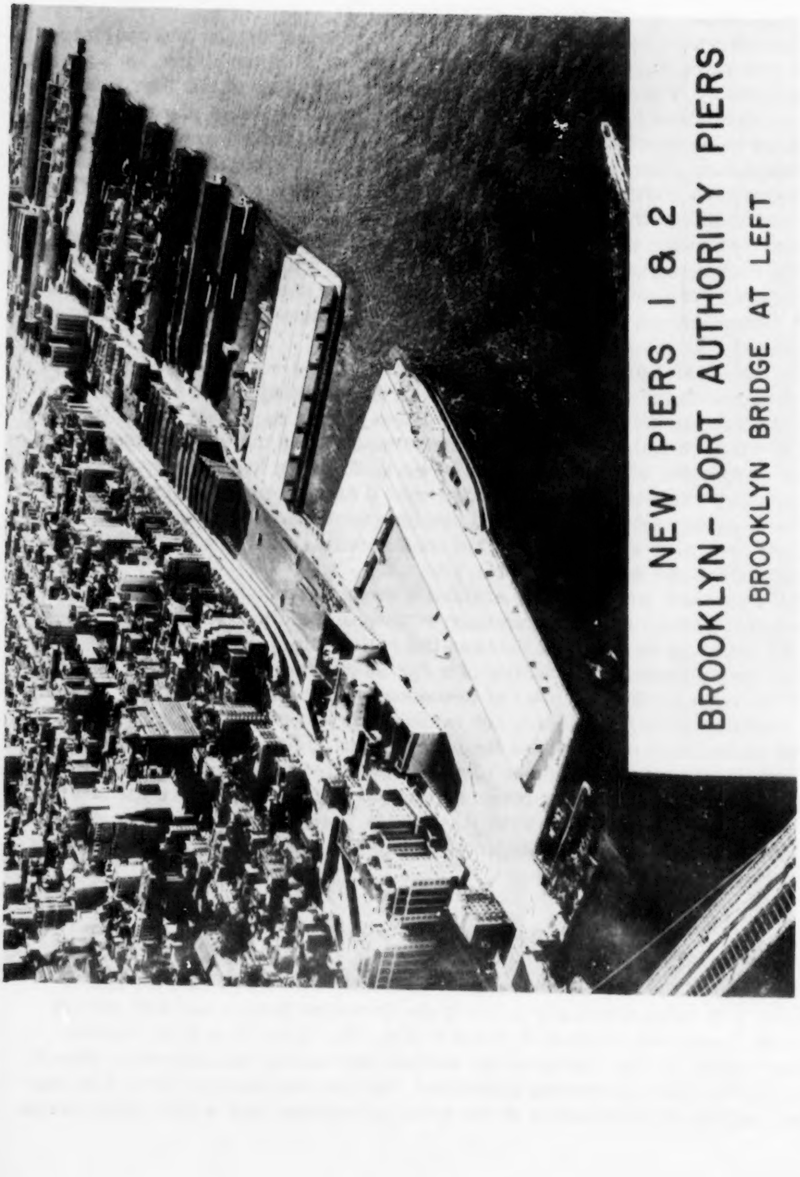

Fig. 16. Brooklyn - P. A. Piers - New Piers 1 & 2 Brooklyn Bridge at Left

Pierhead Line in this area. At the north end the new structure will extend 270 feet out from the bulkhead to the U. S. Pierhead Line, and will then extend towards the south for a length of 1,000 feet. It will then return eastward to the existing bulkhead, a distance of 500 feet, thus completing a trapezoidal shape. This will be a three-berth facility. The north face of the structure will be suitable for use by shallow-draft vessels only.

Again, the basic criteria for this pier call for 90,000 square feet of covered structure per berth with 25' exterior aprons, a shed column spacing of approximately 40' x 58', and a berthing depth of 32' minimum alongside. The deck live load is again 500 lbs. per square foot or the H20-S16 truck loading, and the lateral load is as before. Fireproof construction is again required.

The borings taken at this site showed a condition quite similar to that at the new Pier 2. The area is underlain by up to twenty feet of organic silt, a layer of sand up to twenty feet thick, and up to fifty feet of clay above the bedrock. Rock lies from 80' to 100' below Mean Low Water. These conditions favored the same types of piles considered for Pier 2.

Economic studies were first directed toward the possible reuse of the existing pier substructures. It was apparent that those of Piers 3, 4 and 5 were unsuitable for incorporation into the new structure. Pier 6, however, which was constructed in 1930 by the Board of Transportation, was carefully inspected and the substructure found to be in excellent condition. This pier is constructed on untreated timber piles and has a reinforced concrete beam-and-slab deck. The piles are not fireproofed, but a series of firewalls were provided beneath the deck, and nozzle openings were placed in the deck slab. With the condition of this pier established, and because it spanned the subway tubes, it was decided to retain as much as possible of its substructure for incorporation into the new pier.

In studying the area to the north of Pier 6, it was found that the most economical structure would be a sheet pile bulkhead, anchored into sand fill which would be placed over the area. However, in the area south of Pier 6, fill could not be placed immediately adjacent to the subway tunnels because of the excessive load which would be placed upon them. It was decided therefore to make the construction south of Pier 6 of a pile-supported type. In order to make the construction in that area compatible with that of the existing Pier 6, it was decided that timber piles should be used, driven to 20 tons capacity, with fireproofing as described for Pier 2.

To summarize the construction, then, the pier will consist of sand fill retained by a sheet pile bulkhead to the north of Pier 6, of the substructure of existing Pier 6 modified, and a pile-supported concrete deck south of Pier 6 (Figure 17). Bituminous concrete pavement will be placed on the filled section and a bituminous concrete wearing surface placed on the concrete deck to provide the same floor finish over the entire area.

At the time that the design of the bulkhead portion began, it was considered doubtful that steel sheet piling in sufficient quantity could be obtained in time to meet the required construction schedule. As a result, it was decided that a concrete bulkhead should also be designed and made an alternate to the use of steel. The concrete bulkhead sheet piling would be a T-shaped, pretensioned concrete section with tongue and groove joints, approximately 2'-6" wide. A 10" core would run the full length of the sheeting to reduce its weight. Each sheet would be 67 feet long and weigh approximately 22 tons. These sheets would require jetting in order to place them, since their size

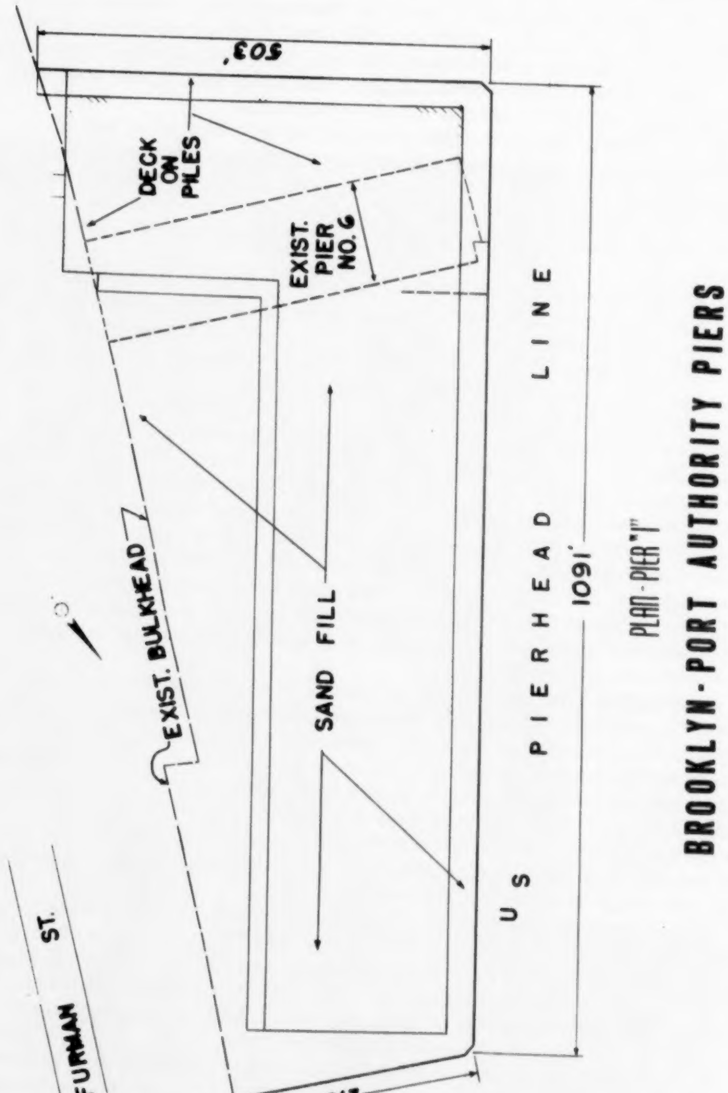

Fig. 17. Brooklyn - P. A. Piers - Pier 1 - Plan

would prohibit driving them into the dense sand layer beneath the site. During the design period, a test program was carried out to establish the feasibility of jetting these sheets into place. Prototype concrete sheets were manufactured and placed successfully at several locations along the line of the bulkhead.

In this concrete bulkhead design, the tie rods would be spaced at 7'-6" centers. Precast, pretensioned concrete walers, each 14'-9" long, would be used at each end of the rods. The tie rods would be placed in pairs, with a waler attached at each end, into slots provided by a shorter sheet pile section placed between the typical sections. This slot would then be closed with a precast section. Pretensioned concrete sheet piling for the anchorage would be 1'-2" thick, 2'-6" wide and 18 feet long. Mooring bitts would be anchored into a cast-in-place concrete cap and would be provided with their own deadman-type anchorage.

The steel bulkhead design would be almost identical in all respects with that described for the Pier 11 project in Atlantic Basin, with a submerged bulkhead waler (Figure 18). The sheet piling would be the MZ or ZP 38 section, 67 feet long, retained by tie rods at 9 foot centers. The walers at each end would be rolled steel shapes made up in lengths of approximately 17'-9". As in the case of Pier 11 it was anticipated that these tie rods and walers would be placed in pairs in slotted or cut sheets to permit placing under water. Again, mooring bitts would be set in a cast-in-place concrete cap and anchored as described above.

In this case, it would again be necessary to provide a cathodic protection system and bitumastic paint on all steel surfaces above the low tide line.

In the case of either bulkhead design it was found necessary to make special provision for anchorage of the bulkhead at the northwest corner of the pier. After considerable study, it was found that the most suitable type of anchorage would be provided by a relieving platform supported on timber piles. The use of the platform reduces the lateral pressure against the bulkhead and makes possible the provision of sufficient batter piles to anchor this corner.

At the junction of the fill area and the north edge of Pier 6, a timber sheet pile bulkhead will be provided to insure the retention of the fill—above the level where it spills beneath the pier.

In order to construct either type of bulkhead, the work required would be as follows: -

1. Demolition of Piers 3, 4 and 5.
2. Dredging of the bulk of the highly compressible river silt.
3. Placing of fill in the area in such a manner as to permit the driving of the sheet pile bulkhead.
4. The construction of the bulkhead itself.

In connection with the placing of fill, it is required by the Corps of Engineers that the entire exposed perimeter of the filled area be surrounded with a rock dyke to retain the fill during construction. As a result, the actual line of initial fill to be placed is determined by that requirement and by the necessary working space required to construct the bulkhead.

The deck of existing Pier 6 will require considerable work to make the deck elevations correspond to those required by the new shed. It will be necessary to open up that deck to drive additional piles at the location of the new columns, and to provide special footings under those columns to spread

Fig. 18. Brooklyn - P. A. Piers - Pier 1 - Bulkhead Area - Typical Cross Section

those loads sufficiently over the existing piles. Directly over the subway tubes, it is necessary to provide a hollow type of structure to regrade the floor and still maintain the loads on the existing piles within acceptable limits. In the areas adjacent to this hollow floor construction it is necessary to place earth fill to regrade the floor.

Again, during the design period, test loads were placed on representative piles to establish their load capacity.

Because of the location of the face of the new pier, it is necessary to reframe the end of Pier 6. In order to avoid any pile driving adjacent to the subway tubes, the existing piles in the area will be reused.

Turning attention now to the area south of Pier 6, it has already been stated that this area would be supported on timber piles, to be fireproofed either with concrete pipe jackets or with the precast concrete pile extensions. During the design period, test piles were driven to verify the assumed 20-ton pile capacity. Here again two alternate deck designs were prepared and advertised for the Contractors' choice. Studies indicated that a cast-in-place deck similar to that described for Pier 2 might be employed, or a precast, pretensioned deck slab could be placed on cast-in-place pile caps. For the cast-in-place deck, it would be possible to use an 8" slab with small haunches over the tops of the piles, or a constant depth slab 12" thick without haunches.

Because of the radically different types of construction to be used in this pier, the areas described were divided into two substructure contracts, one for the bulkhead and one for all deck construction. Bids have been received for both contracts. The successful bidder, Horn Construction Corporation, chose the steel bulkhead design for the area north of Pier 6, and the cast-in-place flat slab deck construction for the area south of Pier 6.

Contracts for the demolition, dredging and fill in the area north of Pier 6 were let earlier this year. At this writing, the demolition and dredging has been completed, and the placing of fill is under way.

The shed for this pier will be the same steel truss type to be utilized on Pier 2, with aluminum roofing and siding. Again, a 4' concrete cheek wall will be provided between the cargo doors. The shed will be an "L" shaped structures, 200' wide with a 16' loading platform along the inshore side. In the area north of Pier 6, the shed will be supported on spread footings. Because of the expected differential settlement between the portion of the building which is pile-supported and that on fill, provision has been made in the roof members to allow for such a possibility.

CONCLUSIONS

It would seem reasonable to conclude from a review of the various waterfront structures which have been described herein that each problem must be approached on the basis of the individual conditions peculiar to its site and requirements. The preparation of alternate designs often may reflect economies in construction by providing means by which various contractors can adapt their equipment and particular techniques more readily to one design than another. New construction techniques, new developments in construction materials and the availability of materials and equipment must be constantly borne in mind in order to effect the greatest possible economy in each new structure.

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WAVE RUN-UP ON ROUGHENED AND PERMEABLE SLOPES[a]

Rudolph P. Savage[1]
(Proc. Paper 1640)

SYNOPSIS

Laboratory tests determining run-up on various beach slopes as a result of wave action are described. Curves relating the run-up to wave steepness, slope roughness, and slope permeability are presented.

INTRODUCTION

Wave run-up is defined as the vertical height of the limit of uprush reached by a wave breaking on a slope, using the still water level as a reference (Fig. 1). Run-up is important because it determines the height to which certain structures must be built to prevent overtopping from waves in order to prevent flooding landward of the structures. Also, artificial sand fills are used extensively for shore protection and beach restoration and wave run-up is important in determining the design berm elevation of these sand fills.

Until the past few years, very little run-up data was available for design purposes. In 1953, Grantham(1)* of the University of California, published run-up data on smooth and permeable slopes of 15, 30, and 45 degrees. Recently, Saville,(2) of the Beach Erosion Board, presented run-up data on a wide range of smooth continuous slopes, smooth composite slopes, and some data on a 1 on 1-1/2 riprap faced slope. However, very little of the foregoing data deal with the effect of slope roughness on wave run-up, and the data on the effect of slope permeability are confined to a narrow range of slopes. This paper presents the results of wave run-up tests made at the Beach Erosion

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a. Presented at the New York Convention of the American Society of Civil Engineers, New York, N. Y., October, 1957.

1. Hydr. Engr., Research Div., Beach Erosion Board, Corps of Engrs., U. S. Dept. of the Army, Washington, D. C.

* Numbers in parentheses refer to references on p. 16.

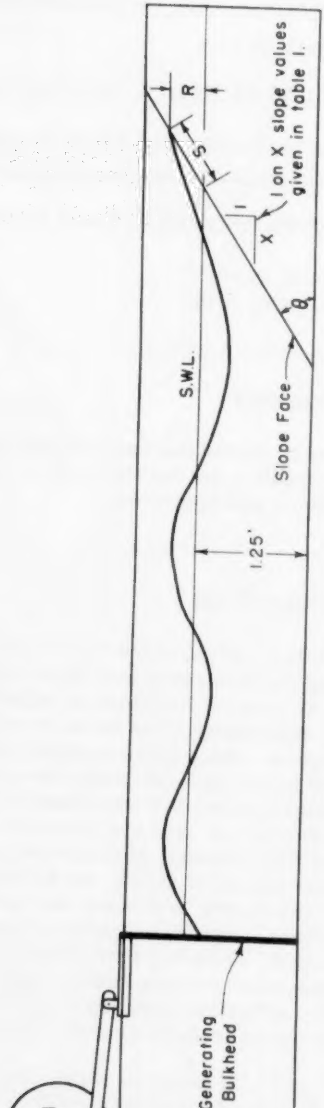


FIGURE 1. SCHEMATIC DIAGRAM OF EXPERIMENTAL SET-UP FOR WAVE RUN-UP TESTS

Board on smooth, roughened and permeable slopes ranging from a relatively gentle slope of 1 on 30 to a vertical wall.

Test Facilities

A schematic diagram of the experimental set-up for these tests is shown in Fig. 1. The wave tank shown is 96 feet long, 2 feet deep and 1-1/2 feet wide. The wave generator is a vertical bulkhead push-pull type generator attached eccentrically by a connecting arm to a driving disk 2 feet in diameter. The driving disk is driven through a varidrive unit by a 2-1/2 hp electric motor. The wave period was changed by varying the gear ratio of the varidrive unit, enabling continuous range of wave periods from 0.5 seconds to 5 seconds to be obtained. The wave height was changed by varying the eccentricity of the connecting arm on the driven disk. This allowed a continuous range of wave heights to be generated varying from very small heights (on the order of 0.001 foot) to the largest stable height for any particular water depth and wave period.

Wave heights were measured with parallel-wire resistance wave gages(3) and recorded on a Brush oscillograph. Accuracy of individual height measurements should be within ± 5 per cent. Water depths were measured with a point gage and vernier which could be read to ± 0.001 foot.

The slopes tested ranged from 1 on 30 to vertical. All of the slopes started at the tank bottom and sloped continuously upward on a constant slope. The slopes tested are listed in Table I along with the various slope materials used to test the effects of roughness and permeability. The slopes were constructed of plywood with the exception of the 1 on 10 and 1 on 30 slopes which were constructed with a base of fine sand covered with a 1/2-inch layer of smooth concrete. The effect of roughness was tested by covering the smooth slopes with a single layer of material (glued to the slope); the effect of permeability was tested on slopes composed entirely of the material to be tested.

The materials used to test the slopes for the effect of roughness and permeability are also listed in Table I. The characteristics of the materials including the coefficients of sorting and skeweness(4) as determined from standard sieve analyses and permeability tests(5) are shown in Fig. 2. Here the permeability is given in $ft.^2$ (a darcy = $1.0624 \cdot 10^{-11} ft.^2$). The 0.2 mm (No. 1) sand was a typical fine beach sand. The 1.0 and 2.0mm (Nos. 2 and 3) sands were well-sorted sands obtained by sieving out the particles finer or coarser than those near the median diameter. The 3.5mm (No. 4) material was a poorly-sorted, well-rounded pea gravel, and the 10.0mm (No. 5) material was a typical crushed stone commonly used as an aggregate in concrete.

Test Procedure

In general the test procedure involved placing a test slope in the end of the wave tank opposite the generator and propagating waves of known characteristics against the slope. The first few waves generated were ignored and the run-up for the next 6 to 15 waves was measured by reading the run-up for each wave by eye on a scale marked on the face of the slope in hundredths of a foot of vertical elevation. Run-up measurements were stopped before reflections from the beach could travel to the generator and back to the beach, and the average of the individual run-up readings was taken as representative of that run.

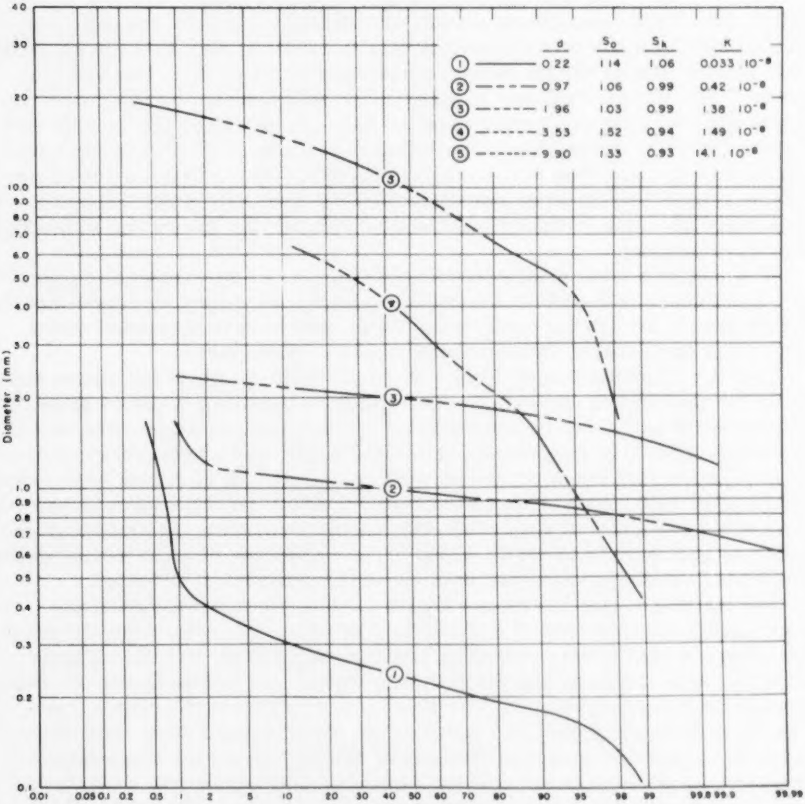


FIGURE 2. CHARACTERISTICS OF MATERIALS USED FOR ROUGHNESS AND PERMEABILITY

A constant water depth of 1.25 feet was used in all the tests since Saville(2) has shown that varying the water depth at the toe of the structure has a negligible effect on the relative wave run-up when the water depth at the toe of the structure is in the order of three times the deep-water wave height or greater.

The wave characteristics were determined by calibrating the wave generator for the 1.25 foot water depth. The wave generator was calibrated by placing a wave absorber in the beach end of the tank and generating a wave train with known and reproducible settings on the generator eccentric and varidrive. The average height of the generated wave train was measured with a parallel-wire gage at six-foot intervals along the tank beginning near the generator. This procedure was repeated for each combination of wave heights and periods used in the tests. These heights were then plotted vs. distance along the tank (Fig. 3) and a smooth curve was drawn through these points. The wave height value obtained from this smooth curve at a distance which coincided with the toe of the test slope was then used as the height value for that particular generator setting and test slope. In this manner, wave heights were determined for all the wave and test slope conditions used in the tests. The wave heights at the toe of the 1 on 30 slope for each of the 7 wave periods used in the tests are shown in Table 2. The wave heights for the steeper slopes are slightly smaller than those in Fig. 2 since the steeper slopes were shorter, providing a longer decay distance before the waves reached the toe of the slope. With the wave heights, water depth, and wave periods known, the other needed wave characteristics were computed using shallow water theory.

After the wave characteristics had been determined, the procedure was to construct the slope to be tested in the beach end of the wave tank. The smooth surface of this slope was then tested with the wave conditions generated by the previously calibrated settings on the wave generator. The surface of the slope was then roughened by painting it with a water resistant glue and spreading a roughness material over the wet glue. After the glue was dry, the excess roughness material was swept away leaving a single layer of roughness material approximately one diameter deep over the face of the slope. This roughened surface was then tested with the same wave conditions that were used on the smooth slope. The first roughness material tested was then removed and another applied and tested until the slope had been tested with the roughness materials shown in Table I.

The permeable slopes were tested in the same manner as the roughened slopes and were composed entirely of the material to be tested with the exception of the 1.0, 2.0 and 3.5mm material on the 1 on 30 slope. There was not enough of these materials available to construct a 1 on 30 slope and, consequently a 5-inch layer of these materials was used over the surface of the 1 on 30 smooth slope. It was felt that this would have essentially the same effect on the wave run-up as a 1 on 30 slope constructed entirely from these materials. To check this, the 10.0mm material was tested using both a 5-inch layer of permeable material over the smooth 1 on 30 slope and a 1 on 30 slope constructed entirely of the 10.0mm material. The results of these tests are shown graphically in Fig. 4. Here the relative run-up (R/H_0' or the wave run-up divided by the deep water wave height)* is plotted against a function of the deep water wave steepness (H_0'/T^2 or the deep water wave height over the wave period squared) for both a 5-inch layer of the 10.0mm material and the

* See p. 17 for symbol definitions.

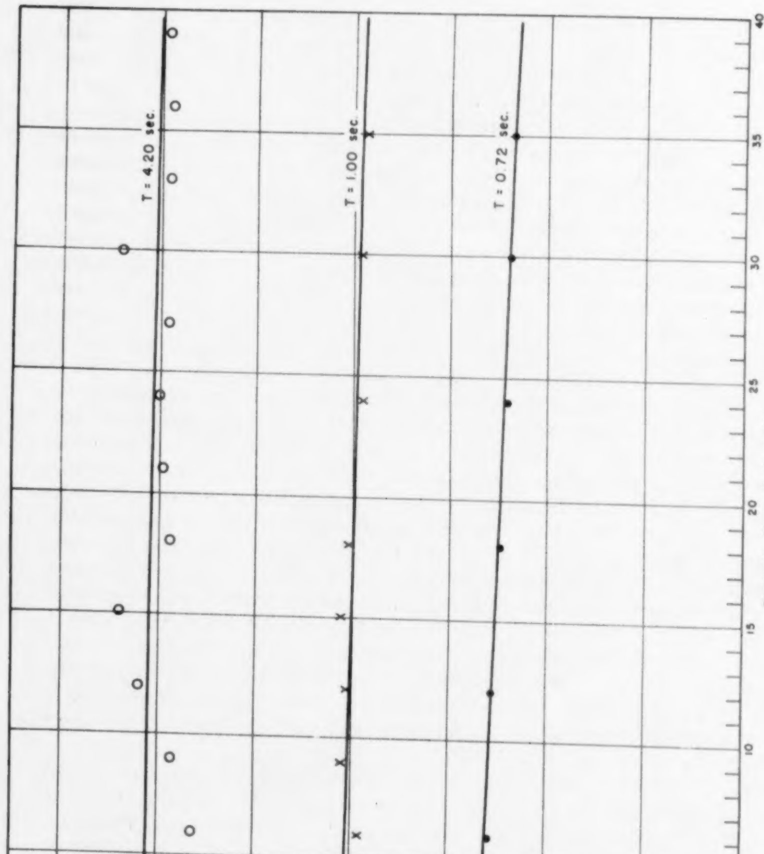


FIGURE 3. REPRESENTATIVE PLOTS OF THE VARIATION OF WAVE HEIGHT WITH DISTANCE ALONG TANK

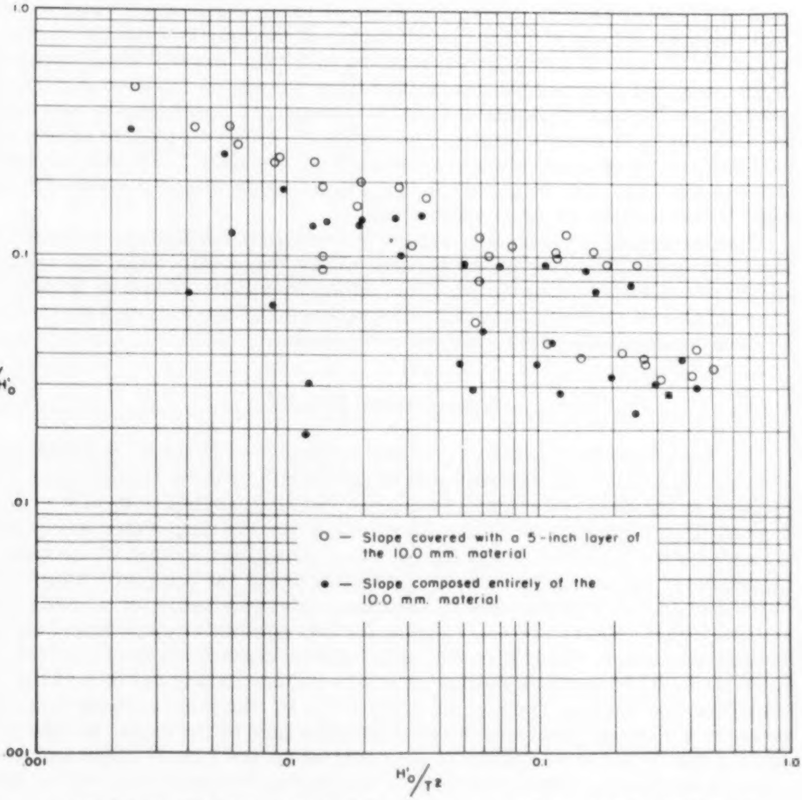


FIGURE 4. RELATIVE WAVE RUN-UP ON A 1 ON 30 SLOPE COMPOSED ENTIRELY OF THE 10.0 mm MATERIAL, COMPARED WITH THE RELATIVE WAVE RUN-UP ON A 1 ON 30 SMOOTH SLOPE COVERED WITH A 5-INCH LAYER OF THE 10.0 mm MATERIAL

slope composed entirely of the 10.0mm material. Although the difference in results is small, it is significant. However, since the 10.0mm material is approximately 10 times as permeable as the 3.5mm material ($14 \cdot 10^{-8}$ft.^{2} compared to $1.5 \cdot 10^{-8}$ft.^{2}) it is believed that the results of the tests with a 5-inch layer of the smaller materials did not vary significantly from what they would have been had a complete slope of these materials been tested.

Essentially, the same problem arose with the steep permeable slopes; that is, what length of permeable structure would be required in front of the back wall of the wave tank for a 1 on 1/2 slope or vertical permeable wall to ensure that the wave run-up would not be a function of the length of the permeable structure as well as the other parameters involved. Several short check tests were made and it was found that for the material having the largest permeability (the 10.0mm material) a structure length of 3 feet behaved essentially as a structure of infinite length. Therefore, all of the permeable slopes steeper than 1 on 6 contained enough material to fill the first 3 feet immediately in front of the rear tank wall, and the slope to be tested sloped downward from a point 3 feet in front of the rear tank wall.

The materials tested in some of the permeability tests would not stand on the steep angle required for the steeper slopes, especially under the agitating action of the waves being tested. This difficulty was overcome by holding the face of the test slope in place with a wire screen having a mesh slightly smaller than the diameter of the material being tested.

Experimental Results

The data from the smooth slope tests were plotted in the form shown in Fig. 5. This is a dimensionless plot of the relative run-up (R/H_o') vs. a function of the deep water wave steepness, (H_o'/T^2). (Actually $H_o'/T^2 = 5.12\ H_o'/L_o$, the deep water wave steepness, since the deep water wave length, $L_o = gT^2/2\pi = 5.12T^2$). From Fig. 5, some idea of the scatter of the data can be obtained. In general, the scatter is rather small for the gentle slopes and increases as the slope increases, becoming relatively large for slopes steeper than 1 on 4. There are two reasons for this particular variation of the data scatter with slope. First it is estimated that a constant error of $\pm$ 0.05 ft. could probably be made in reading S, or the distance along the face of the slope that the waves surged up the slope (Fig. 1). Since S varies as $R/\sin\theta$, where θ is the angle between the profile of the face of the slope, and the horizontal (Fig. 1) S, for a constant R, was largest on the 1 on 30 slope and decreased as the slope steepened. Thus, in general, the per cent error in the run-up readings was smallest for the 1 on 30 slope and increased as the slope steepened. Second, observations during the tests indicated that the magnitude of the run-up depended rather critically on the form of the breaker which, in turn, depended on the behavior and timing of the backwash from the preceding wave. These factors appeared to be more erratic on the steeper slopes and could account for much of the erratic scatter of the data from the steeper slopes.

After the smooth slope data had been plotted in the form shown in Fig. 5, a smooth curve was drawn by eye through the points for each slope. From these curves, a composite graph (Fig. 6) was drawn which shows the effect of slope on the relative run-up for isolines of H_o'/T^2. From this graph several conclusions can be drawn. The most important of these are: first, for any

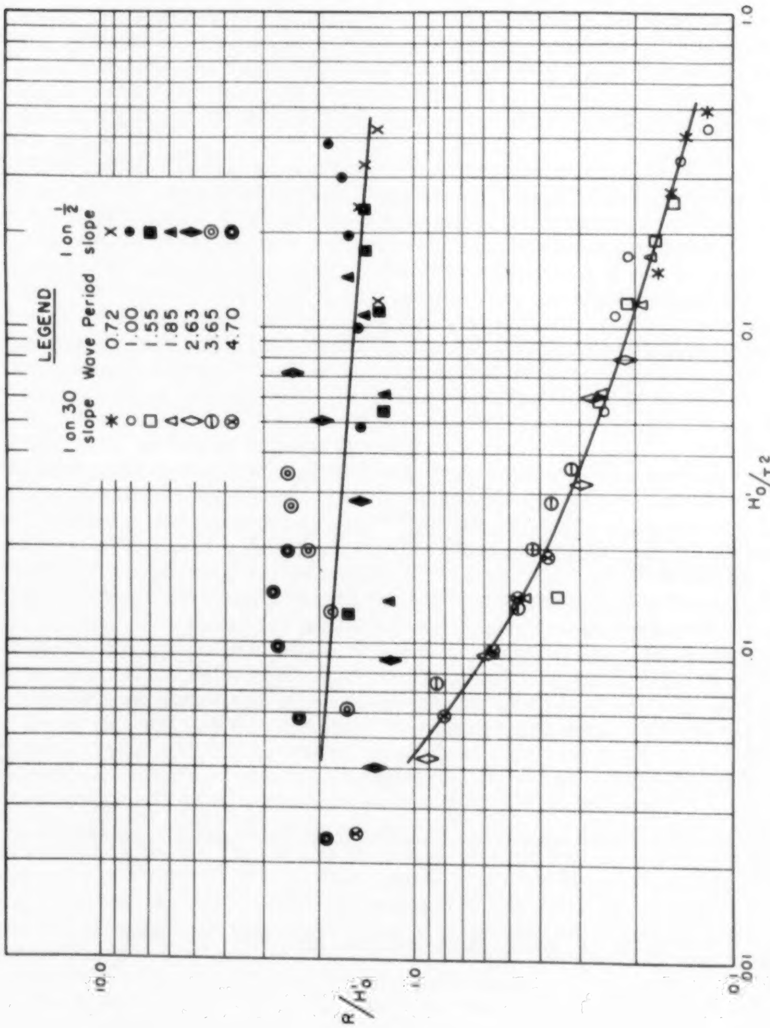


FIGURE 5. RUN-UP ON SMOOTH SLOPES

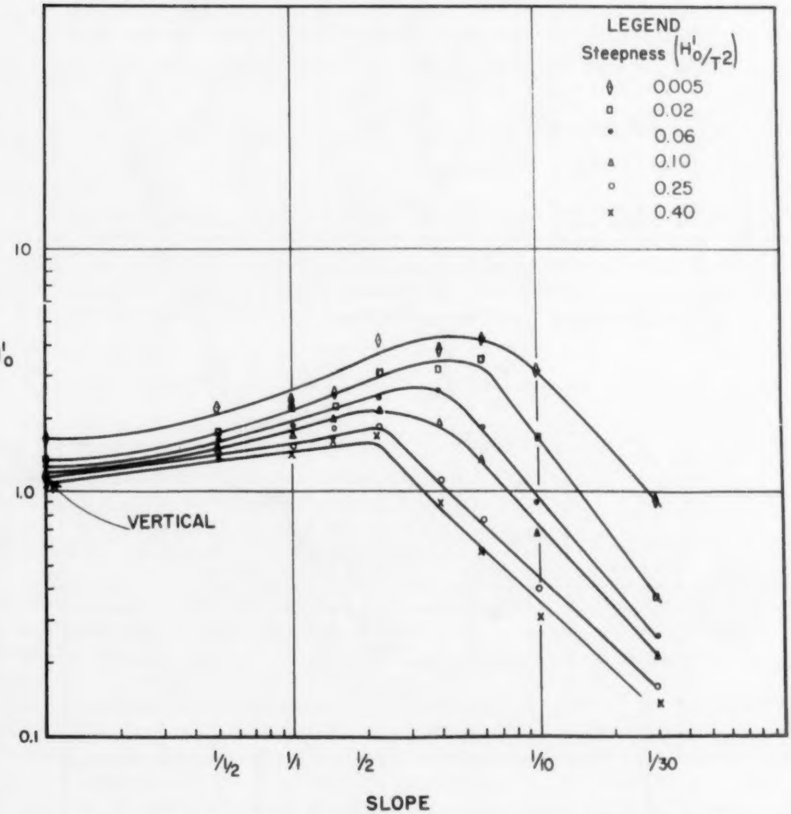


FIGURE 6. WAVE RUN-UP ON SMOOTH SLOPES.

particular slope, the relative run-up increases as the wave steepness decreases; second, for very steep waves, the relative run-up is highest for a slope in the order of 1 on 2; and third, for waves of low steepness the relative run-up is highest for a slope in the order of 1 on 5.

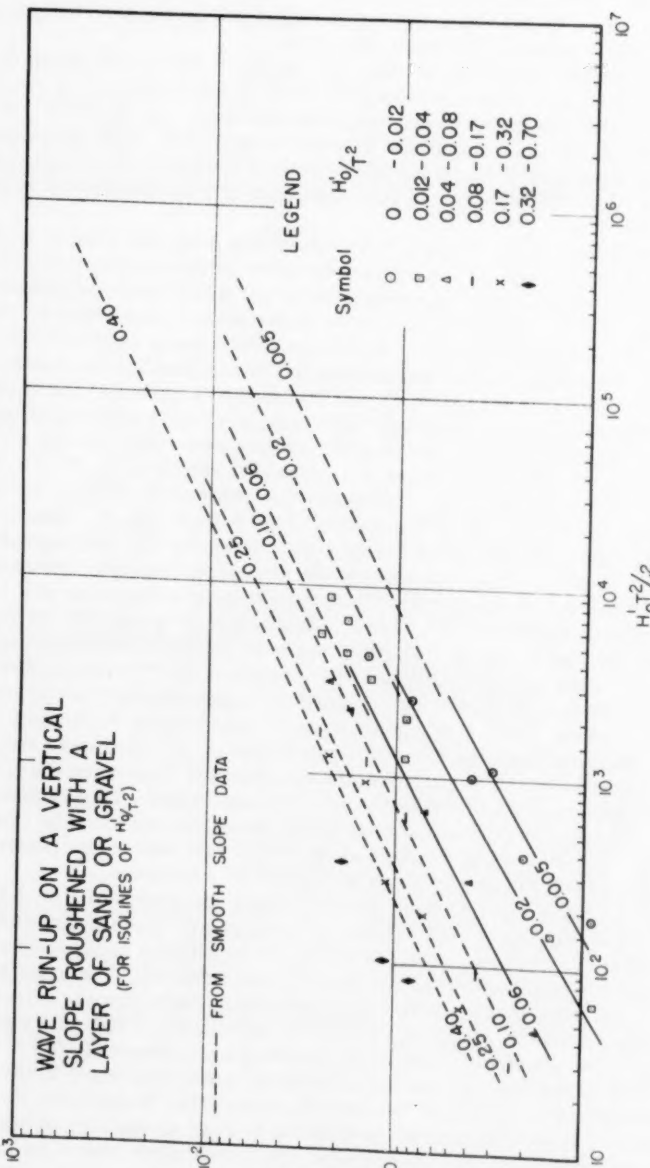
The data from the wave run-up tests on roughened slopes are shown in Figs. 7-15. These are dimensionless plots of R/d, where R is wave run-up and d is the median diameter of the roughness material, vs. $H_o'T^2/d^2$, where H_o' is the deep water wave height and T is the wave period. The parameter R/d gives the wave run-up in terms of the median diameter of the roughness material and $H_o'T^2/d^2$ functions as the reciprocal of a dimensionless roughness coefficient.

The dashed lines shown in Figs. 7-15 were taken from the smooth slope data (Fig. 6). Their position and slope were obtained by assuming that for some large value of $H_o'T^2/d^2$, the roughness of the slope would no longer have a measurable effect on wave run-up. This assumption is supported by the fact that the 0.2mm sand caused no significant reduction in wave run-up on any of the slopes tested. The 0.2mm diameter or any smaller diameter, could then be used with combinations of H_o' and T to compute values of the roughness coefficient which would be essentially equivalent to smooth slope conditions. The values of H_o' and T used in these computations were chosen to be such that in combination they represented the particular values of H_o'/T^2 given for each line on each of the figures. The particular value of H_o'/T^2 for any slope then gives a particular value of the smooth slope R from Fig. 6. This R and the assumed value of d then gives the R/d parameter for the corresponding roughness coefficient. Thus, each of the smooth slope lines was determined by computing two points for any H_o'/T^2 and slope using a diameter of 0.2mm and run-up values obtained from Fig. 6. Since the run-up data for the roughened slopes apparently varied with H_o'/T^2 as well the parameters plotted in Figs. 7-15, the data was separated into the groups of H_o'/T^2 values shown in the figures. The values for the curves used are approximately in the center of the H_o'/T^2 groups used. The curves were drawn by using the smooth slope lines as a reference when the R/d for a particular H_o'/T^2 group was equal to or larger than the smooth slope value and the data was used when the R/d given by the data was smaller than the smooth slope value. This process gives a group of H_o'/T^2 curves for each slope which are the same as the smooth slope curves for large values of $H_o'/T^2/d^2$ and become increasingly smaller than the smooth slope values as $H_o'/T^2/d^2$ decreases.

The H_o'/T^2 groups fall into such narrow ranges for slopes of 1 on 4 and shallower (Figs. 12, 13, 14, and 15) that no attempt was made at drawing a curve for each H_o'/T^2 group. The plotted points are shown according to their H_o'/T^2 group, but only the curves for the 0.25 and 0.005 H_o'/T^2 groups are shown. It is believed that the 0.25 curve adequately describes the effect of waves of high steepness, where $H_o'/T^2 \geq 0.02$ and that the 0.005 curve adequately describes the effect of waves of low steepness, where $H_o'/T^2 < 0.02$.

The scatter of the run-up data for roughened slopes is subject to the same factors discussed previously for the smooth slope data. In addition, the run-up on roughened slopes became more difficult to read as the size of the roughness material increases. For this reason, the data scatter should be larger for the roughened slopes, particularly the conditions utilizing the larger roughness materials.

The data from the run-up tests on roughened slopes as shown in Figs. 7-15 show that in general the effect of slope roughness on wave run-up increases



WAVE RUN-UP ON A VERTICAL SLOPE ROUGHENED WITH A LAYER OF SAND OR GRAVEL
(FOR ISOLINES OF H_o'/T^2)
----- FROM SMOOTH SLOPE DATA
LEGEND
Symbol H_o'/T^2
o 0 -0.012
□ 0.012 - 0.04
△ 0.04 - 0.08
— 0.08 - 0.17
x 0.17 - 0.32
♦ 0.32 - 0.70
0.40
0.25
0.10
0.06
0.02
0.005
R/d
$H_o'T^2/d^2$

FIGURE 7

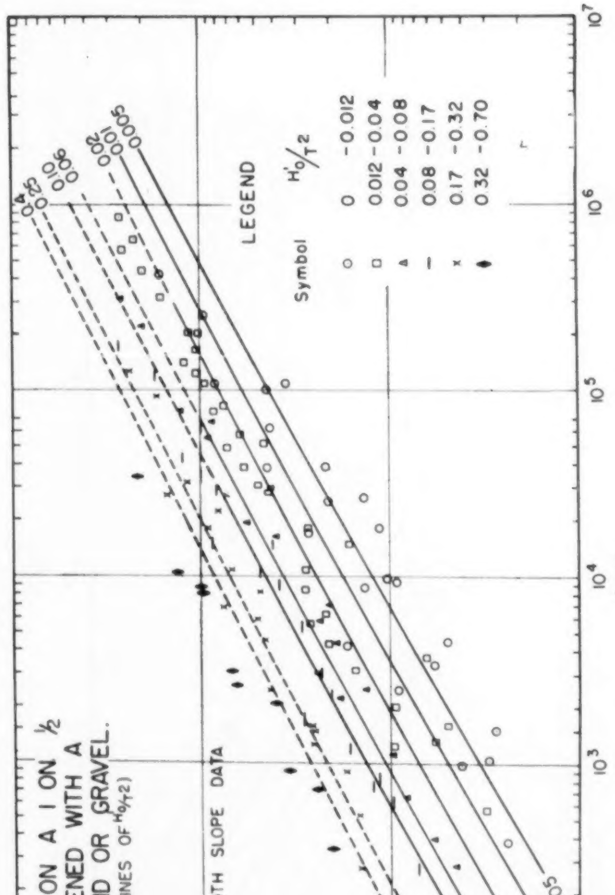

WAVE RUN-UP ON A 1 ON ½ SLOPE ROUGHENED WITH A LAYER OF SAND OR GRAVEL.
(FOR ISOLINES OF H'o/T2)
- - - FROM SMOOTH SLOPE DATA
LEGEND
Symbol H'o/T2
○ 0 -0.012
□ 0.012 -0.04
▲ 0.04 -0.08
— 0.08 -0.17
x 0.17 -0.32
♦ 0.32 -0.70
R/d
H'oT²/d²
0.40
0.25
0.10
0.06
0.02
0.01
0.005

FIGURE 8

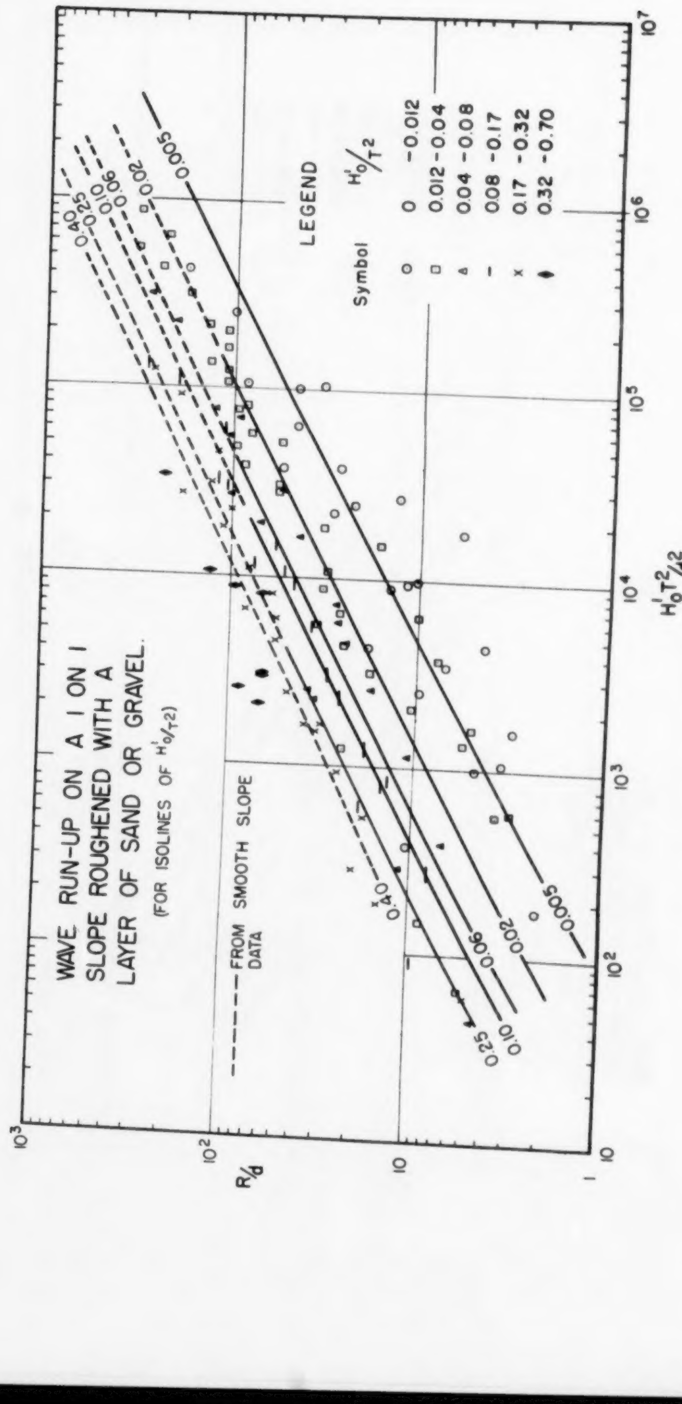

WAVE RUN-UP ON A 1 ON 1
SLOPE ROUGHENED WITH A
LAYER OF SAND OR GRAVEL.
(FOR ISOLINES OF H_0'/T^2)
FROM SMOOTH SLOPE DATA
LEGEND
Symbol
H_0'/T^2
-0.012
0.012 -0.04
0.04 -0.08
0.08 -0.17
0.17 -0.32
0.32 -0.70
0.40
0.25
0.10
0.06
0.02
0.005
R/d
$H_0'T^2/d^2$
10^3
10^2
10
1
10^2
10^3
10^4
10^5
10^6
10^7

FIGURE 9.

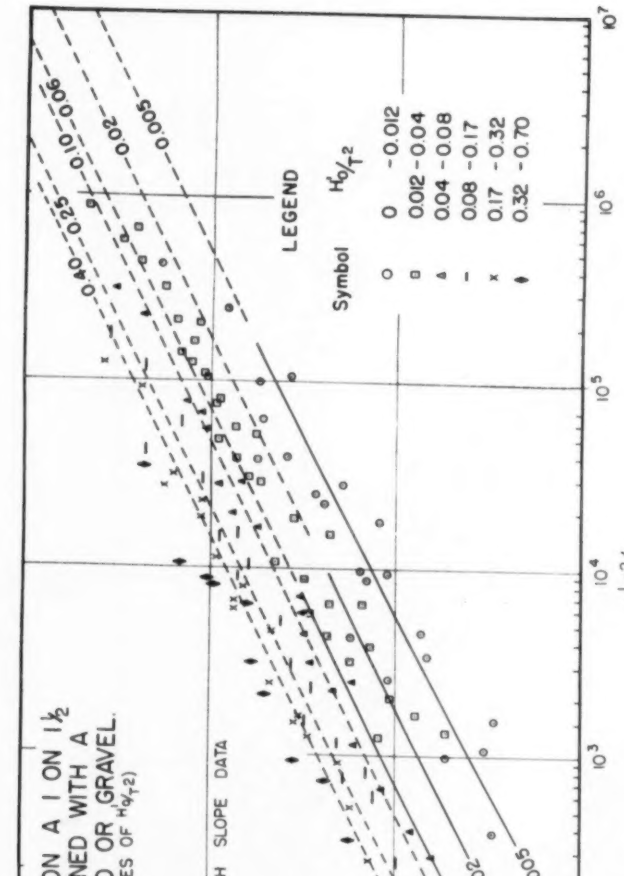

WAVE RUN-UP ON A 1 ON 1½ SLOPE ROUGHENED WITH A LAYER OF SAND OR GRAVEL.
(FOR ISOLINES OF H'_o/T^2)
FROM SMOOTH SLOPE DATA
LEGEND
Symbol
H'_o/T^2
0 -0.012
0.012 -0.04
0.04 -0.08
0.08 -0.17
0.17 -0.32
0.32 -0.70
0.40
0.25
0.10
0.06
0.02
0.005
R/d
$H'_o T^2/d^2$
10
10^2
10^3
10^4
10^5
10^6
10^7
1

FIGURE 10.

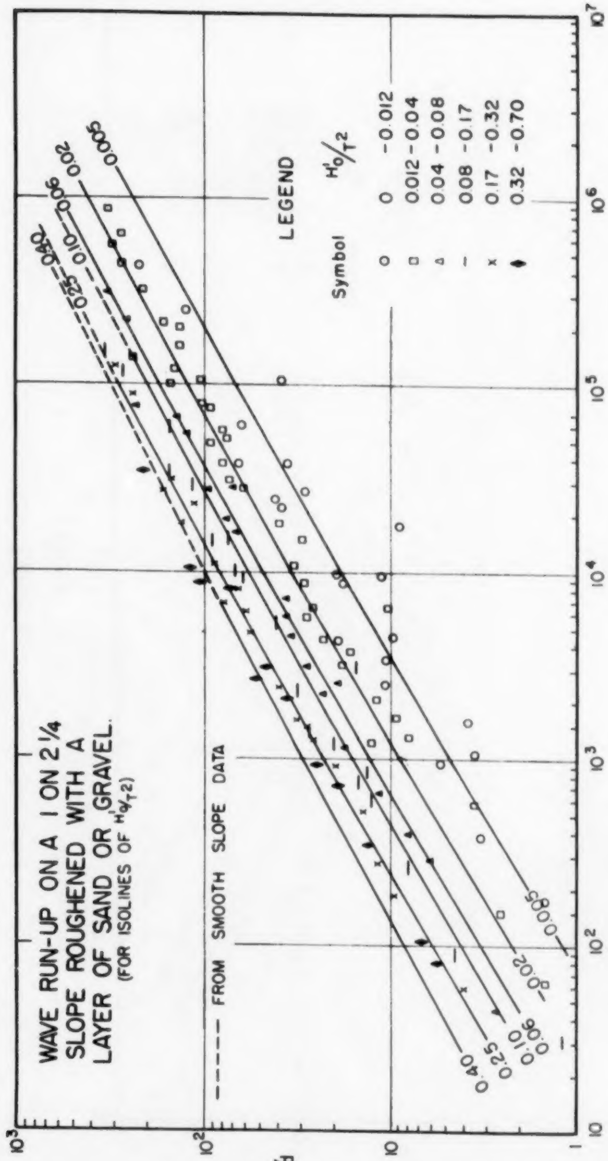

WAVE RUN-UP ON A 1 ON 2 1/4
SLOPE ROUGHENED WITH A
LAYER OF SAND OR GRAVEL.
(FOR ISOLINES OF H'o/T2)
---- FROM SMOOTH SLOPE DATA
LEGEND
Symbol
H'o/T2
0 -0.012
0.012 -0.04
0.04 -0.08
0.08 -0.17
0.17 -0.32
0.32 -0.70
R/d
H'oT2/d2
0.40
0.25
0.10
0.06
0.02
0.005

FIGURE 11.

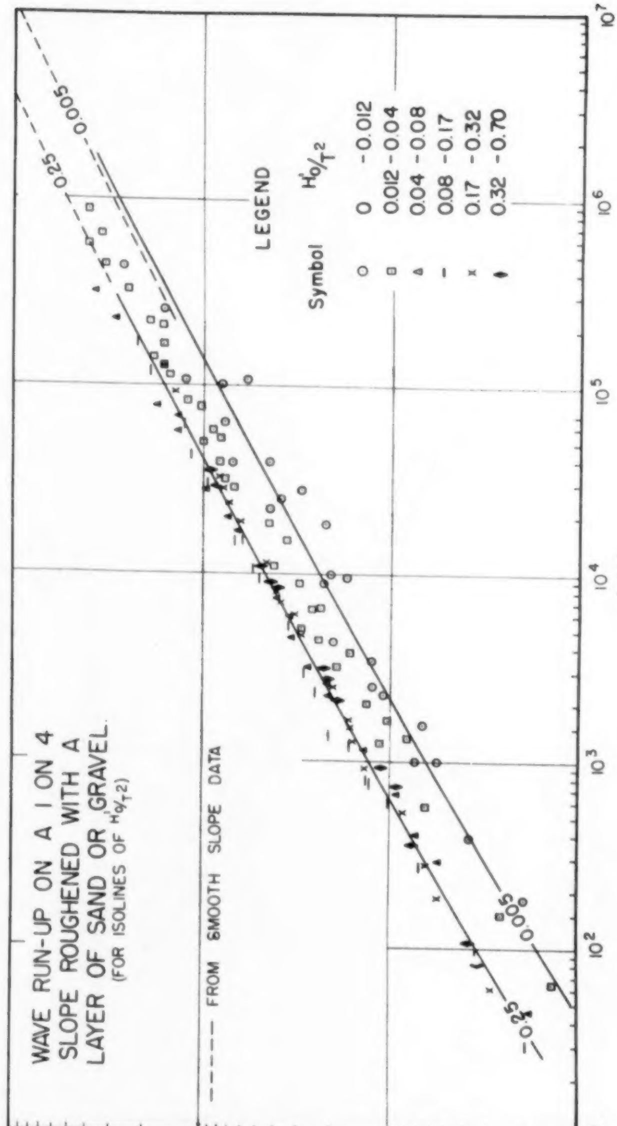

WAVE RUN-UP ON A 1 ON 4
SLOPE ROUGHENED WITH A
LAYER OF SAND OR GRAVEL
(FOR ISOLINES OF H'o/T2)
FROM SMOOTH SLOPE DATA
LEGEND
Symbol H'o/T2
O -0.012
□ 0.012 -0.04
△ 0.04 -0.08
— 0.08 -0.17
x 0.17 -0.32
♦ 0.32 -0.70
0.25
0.005
R/d
H'oT2/d2
10
10^2
10^3
10^4
10^5
10^6
10^7
1
10
10^2
10^3

FIGURE 12.

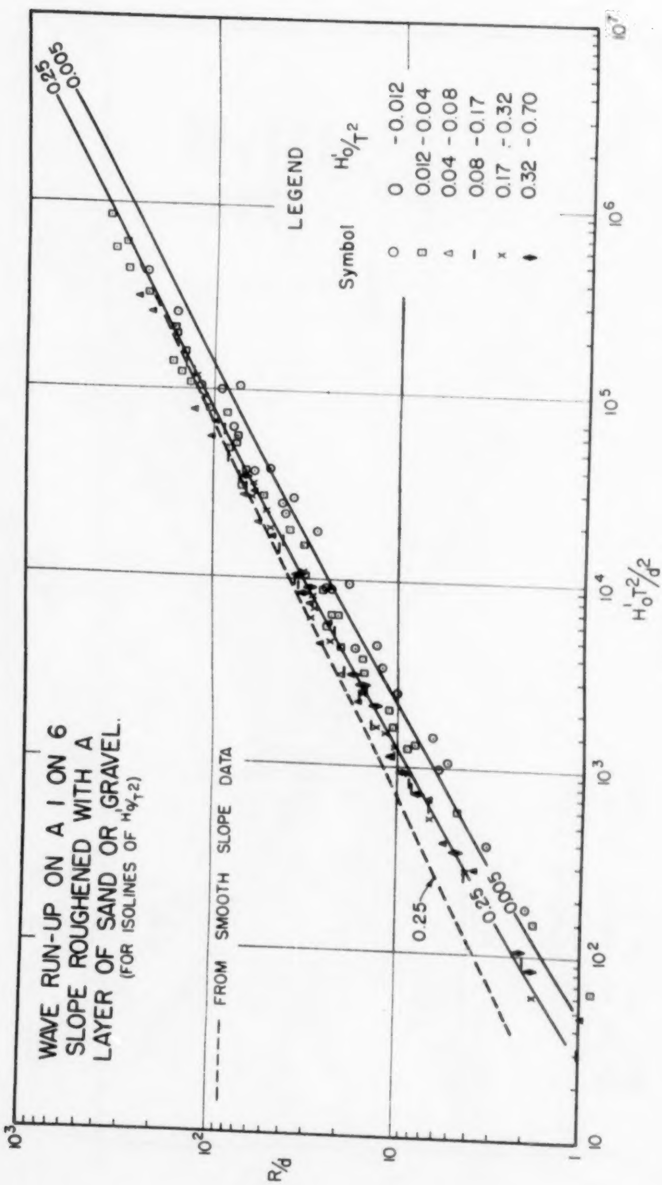

WAVE RUN-UP ON A 1 ON 6 SLOPE ROUGHENED WITH A LAYER OF SAND OR GRAVEL.
(FOR ISOLINES OF H'_o/T^2)
FROM SMOOTH SLOPE DATA
LEGEND
Symbol
H'_o/T^2
O 0 -0.012
□ 0.012 -0.04
Δ 0.04 -0.08
— 0.08 -0.17
x 0.17 -0.32
♦ 0.32 -0.70
0.25
0.005
R/d
H'_oT^2/d^2

FIGURE 13.

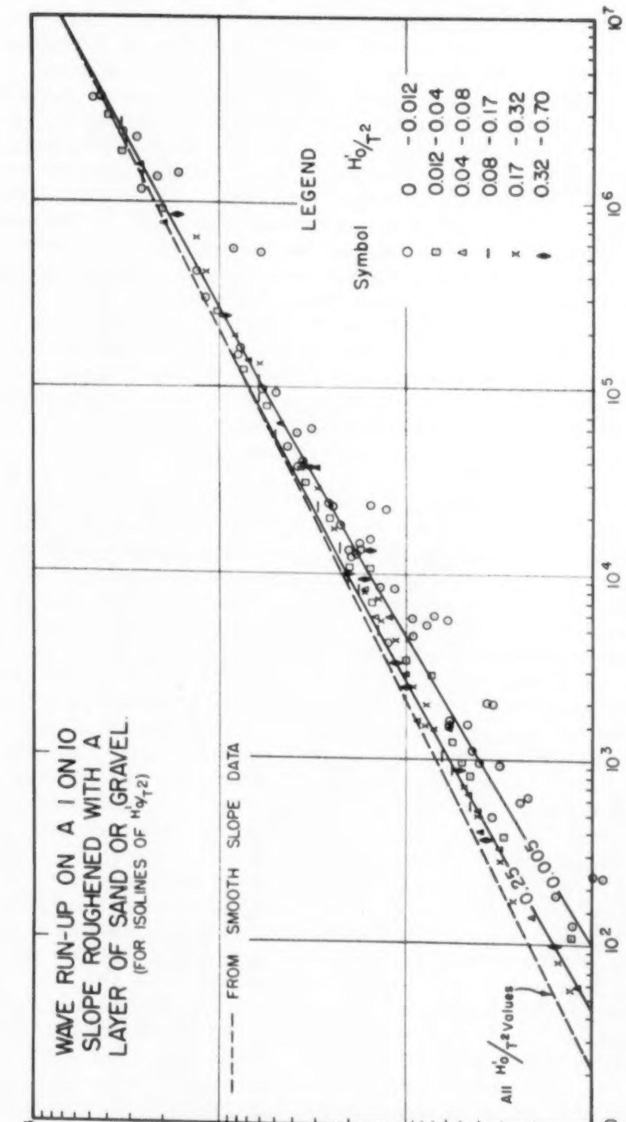

WAVE RUN-UP ON A 1 ON 10 SLOPE ROUGHENED WITH A LAYER OF SAND OR GRAVEL.
(FOR ISOLINES OF H'_0/T^2)
FROM SMOOTH SLOPE DATA
All H'_0/T^2 Values
0.25
0.005
LEGEND
Symbol H'_0/T^2
0 -0.012
0.012 -0.04
0.04 -0.08
0.08 -0.17
0.17 -0.32
0.32 -0.70
R/d
H'_0T^2/d^2
1
10
10^2
10^3
10^4
10^5
10^6
10^7

FIGURE 14.

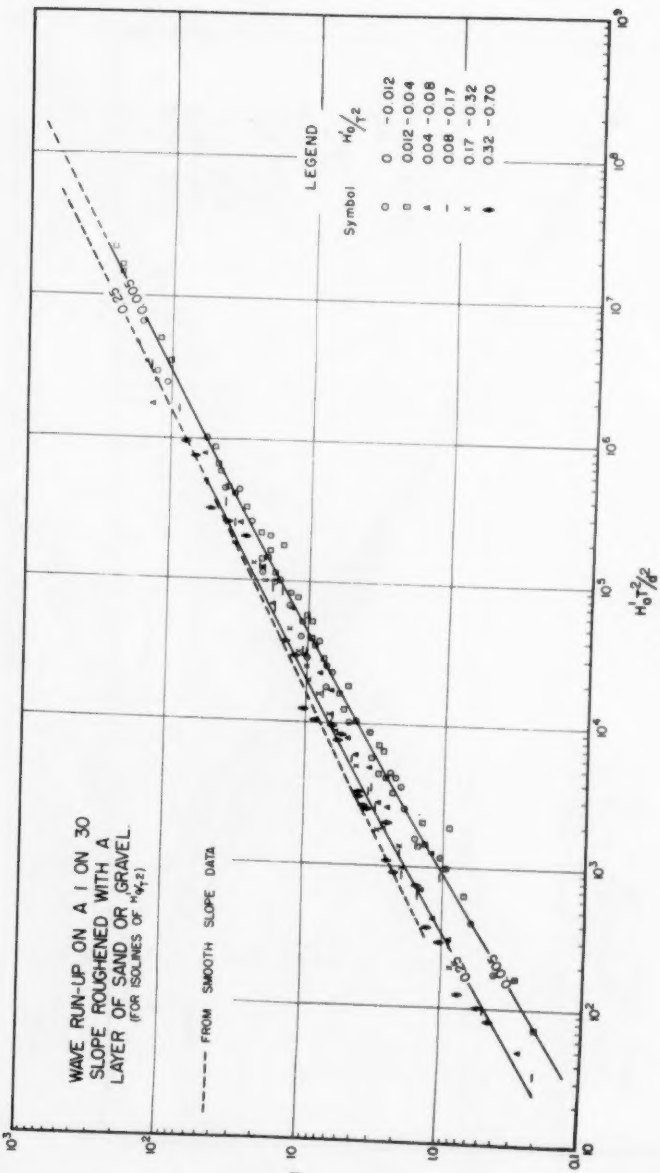

WAVE RUN-UP ON A 1 ON 30 SLOPE ROUGHENED WITH A LAYER OF SAND OR GRAVEL.
(FOR ISOLINES OF H'o/T²)
---- FROM SMOOTH SLOPE DATA
LEGEND
Symbol
H'o/T²
0 -0.012
0.012 -0.04
0.04 -0.08
0.08 -0.17
0.17 -0.32
0.32 -0.70
0.25
0.005
R/d
H'oT²/d²

FIGURE 15.

as the parameter $H_o'T^2/d^2$ decreases and that, for a constant $H_o'T^2/d^2$ and slope, the effect of slope roughness on wave run-up increases as the wave steepness or H_o'/T^2 decreases. Also, for a constant H_o'/T^2 and $H_o'T^2/d^2$, the effect of slope roughness on run-up decreases as the slope steepens.

The curves shown in Figs. 16 and 17 were taken from Figs. 7-15 and show more clearly the effect of slope on the wave run-up on roughened slopes. Here again, as shown by the smooth slope data, it is apparent that the largest run-up occurs on a slope in the order of 1 on 6 for waves of low steepness and a slope in the order of 1 on 2 for waves of high steepness.

The effect of slope permeability on wave run-up for the various slopes is shown in Figs. 18-26. These are dimensionless plots of R^2/K (where K is the permeability of the roughness material in ft.^{2} at 68° F) vs. $H_o'T^2/K$. These plots have the same general form as those used to show the effect of slope roughness; the smooth slope curves shown were obtained in the same manner; the curves were drawn in the same manner, and in general, the results are the same. The most significant difference between the two sets of data is that the effect of slope permeability on wave run-up is more pronounced than the effect of slope roughness. This is logical since the data for the effect of slope permeability on wave run-up as shown in Figs. 18-26 in reality show both the effect of slope roughness and slope permeability since the surface of the permeable slopes was composed of the same roughness materials used in the roughness tests.

Fig. 27 is a comparison of the smooth slope laboratory results with run-up values measured in the 635-foot wave tank at the Beach Erosion Board. This tank is 15 feet wide, 20 feet deep, and 635 feet long, and is equipped with a vertical bulkhead push-pull type wave generator capable of generating waves with periods up to 16 seconds and heights up to 6 feet. The prototype run-up data shown in Fig. 27 were obtained in connection with an experimental program on the equilibrium profile of beaches conducted in this tank. In the equilibrium profile tests, a wave train of constant period and height was allowed to impinge on an initially constant 1 on 15 sand slope (the 0.2mm sand described in Fig. 2) until the slope profile had attained its equilibrium shape for that wave train. The run-up measurements were taken at the beginning of each of four tests and represent wave periods ranging from 3.75 to 16 seconds and heights ranging from 2.0 to 5.3 feet. Since the smallest $H_o'T^2/d^2$ for these runs would be approximately 10^7 and the smallest $H_o'T^2/K$ would be approximately 10^{14}, it appears from the results shown in Figs. 16, 17, 25, and 26 that the effect of roughness or permeability would be negligible. For this reason, the data are compared with the smooth 1 on 15 slope data taken from Fig. 6 and represented in Fig. 27 by the smooth curve. The prototype data indicated slightly less run-up but in general agree with the smooth slope laboratory data.

Fig. 28 compares the wave run-up on roughened and permeable slopes (R in this figure) with the run-up on smooth slopes (R_s in this figure) for comparable slope and H_o'/T^2 values. Since the effect of roughness and permeability varies with the $H_o'T^2/d^2$ and $H_o'T^2/K$ values, constant values of $H_o'T^2/d^2 = 10^3$ and $H_o'T^2/K = 10^7$ (Figs. 7-25) were chosen for the comparison. The curves given in Fig. 28 show that the per cent reduction in wave run-up varies with the H_o'/T^2 value and the slope; decreasing as the H_o'/T^2 value increases and as the slope steepens.

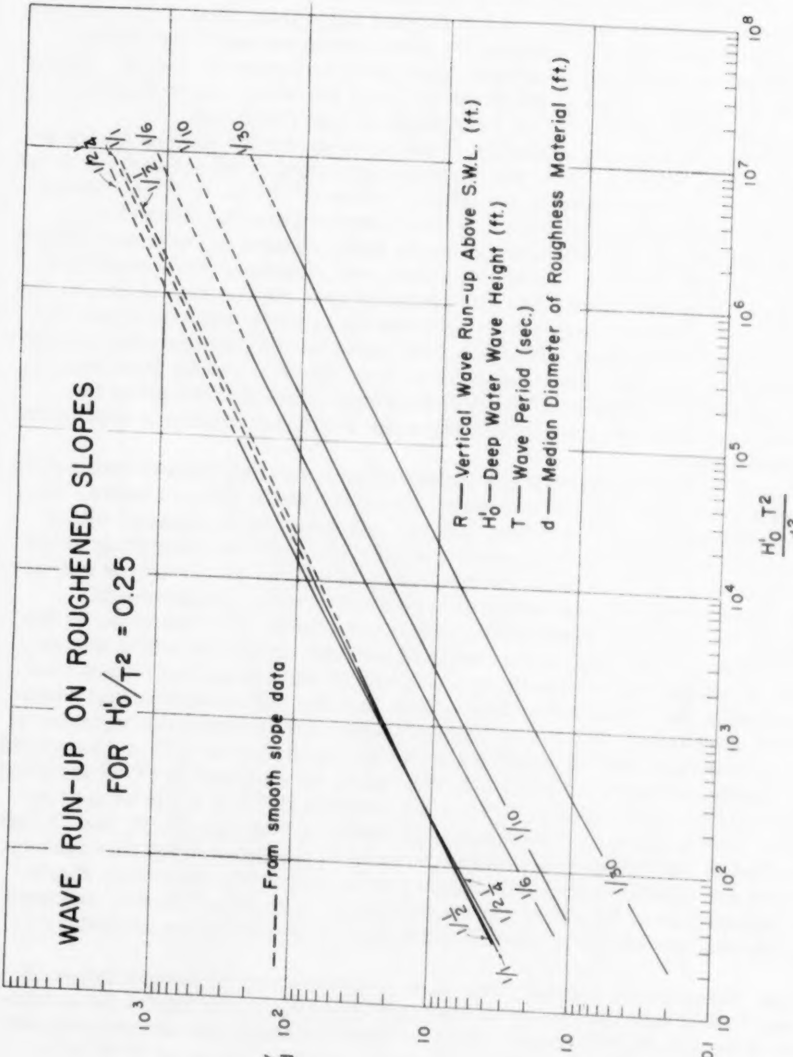

WAVE RUN-UP ON ROUGHENED SLOPES
FOR $H'_o/T^2 = 0.25$
- - - - From smooth slope data
R — Vertical Wave Run-up Above S.W.L. (ft.)
H'_o — Deep Water Wave Height (ft.)
T — Wave Period (sec.)
d — Median Diameter of Roughness Material (ft.)
R/d
$\frac{H'_o T^2}{d^2}$
1/1
1/1½
1/2¼
1/6
1/10
1/30
0.1
1.0
10
10^2
10^3
10
10^2
10^3
10^4
10^5
10^6
10^7
10^8

FIGURE 16.

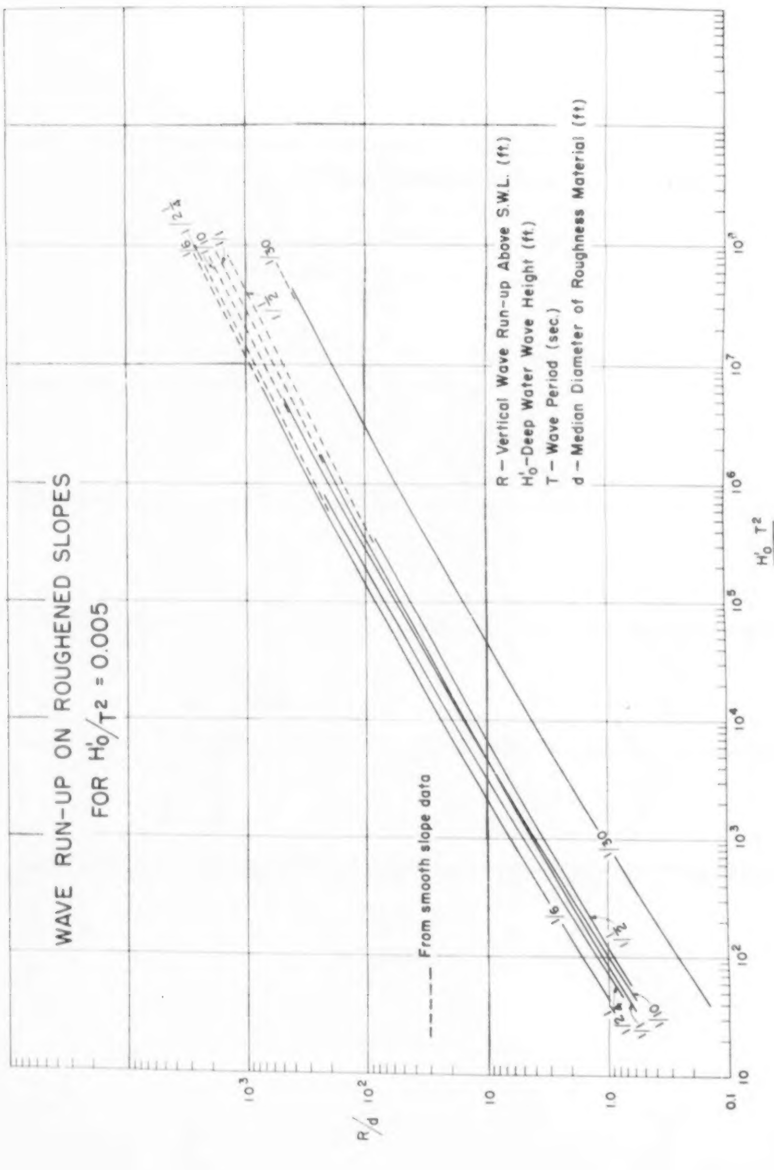
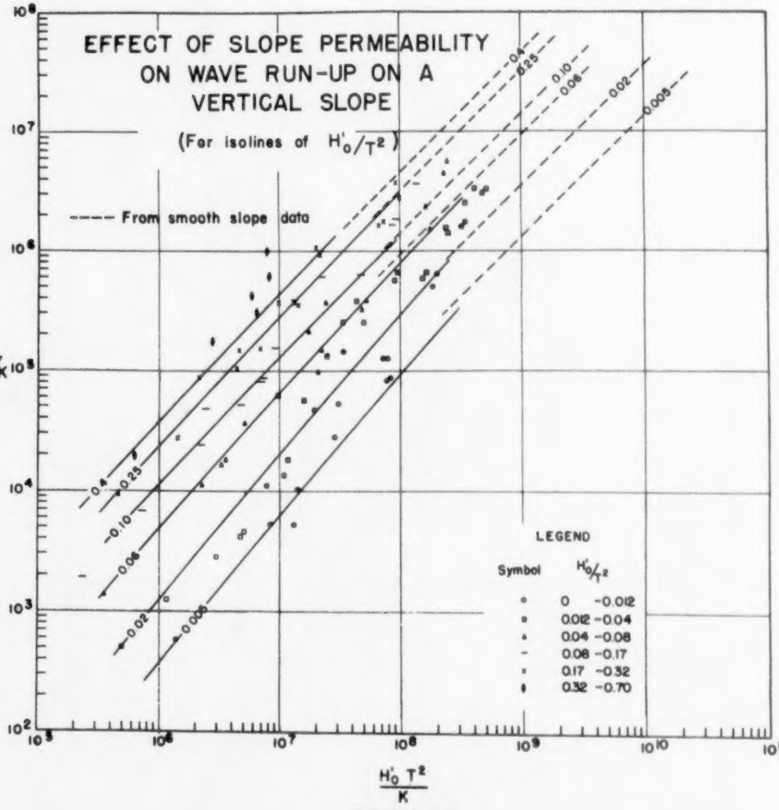

WAVE RUN-UP ON ROUGHENED SLOPES
FOR H'o/T² = 0.005
R – Vertical Wave Run-up Above S.W.L. (ft.)
H'o–Deep Water Wave Height (ft.)
T – Wave Period (sec.)
d – Median Diameter of Roughness Material (ft)
From smooth slope data
R/d
H'o T²/d²
1/6
1/10
1/1
1/1½
1/2½
1/30
0.1
1.0
10
10²
10³
10⁴
10⁵
10⁶
10⁷

FIGURE 17.



EFFECT OF SLOPE PERMEABILITY ON WAVE RUN-UP ON A VERTICAL SLOPE
(For isolines of H'_0/T^2)
---- From smooth slope data
R^2/K
$\frac{H'_0 T^2}{K}$
LEGEND
Symbol H'_0/T^2
0 -0.012
0.012 -0.04
0.04 -0.08
0.08 -0.17
0.17 -0.32
0.32 -0.70

FIGURE 18.

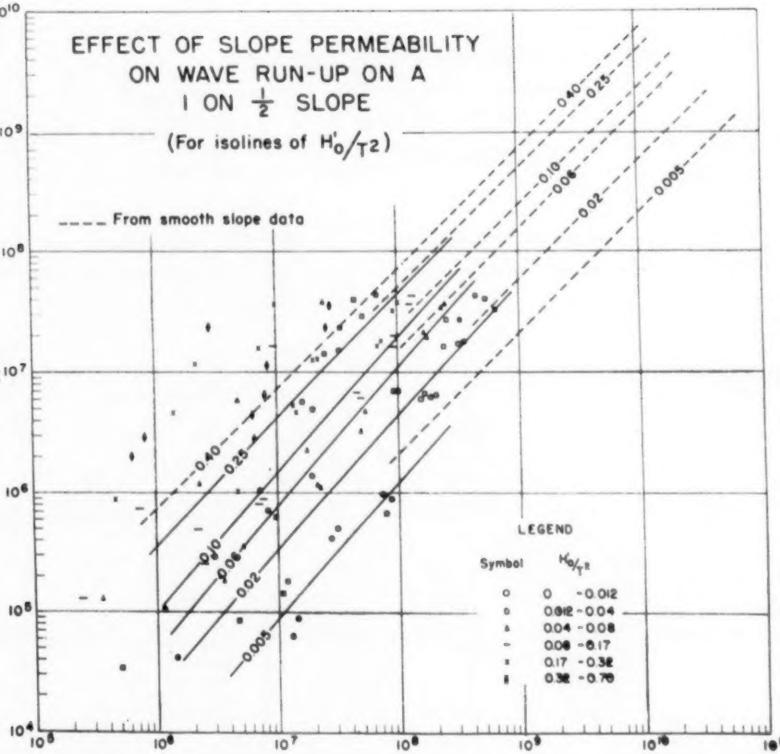
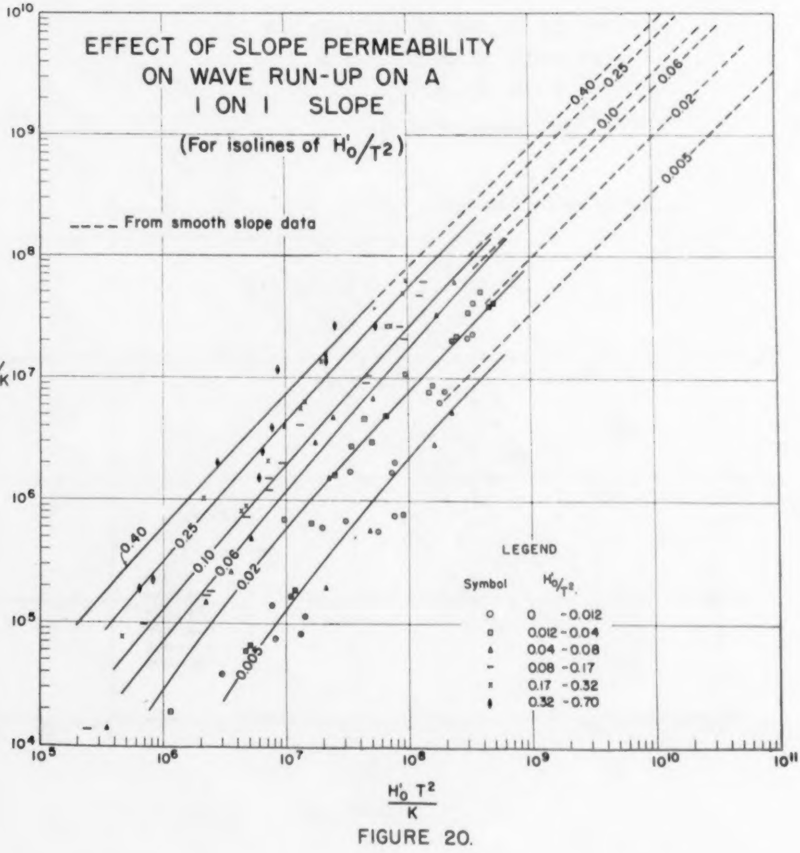
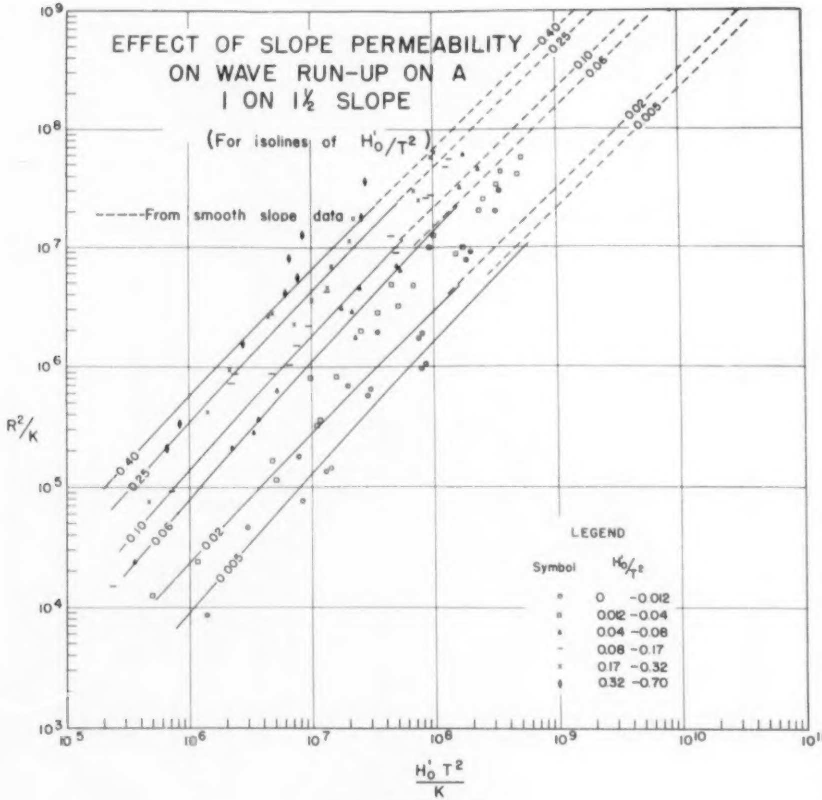

EFFECT OF SLOPE PERMEABILITY
ON WAVE RUN-UP ON A
1 ON 1/2 SLOPE
(For isolines of H'o/T2)
---- From smooth slope data
0.40
0.25
0.10
0.06
0.02
0.005
R²/K
H'o T²/K
LEGEND
Symbol H'o/T²
0 -0.012
0.012 -0.04
0.04 -0.08
0.08 -0.17
0.17 -0.32
0.32 -0.70

FIGURE 19



EFFECT OF SLOPE PERMEABILITY
ON WAVE RUN-UP ON A
1 ON 1 SLOPE
(For isolines of H'_0/T^2)
---- From smooth slope data
0.40
0.25
0.10
0.06
0.02
0.005
R^2/K
10^4
10^5
10^6
10^7
10^8
10^9
10^{10}
10^{11}
LEGEND
Symbol
H'_0/T^2
0 -0.012
0.012-0.04
0.04 -0.08
0.08 -0.17
0.17 -0.32
0.32 -0.70
$\frac{H'_0 T^2}{K}$

FIGURE 20.



EFFECT OF SLOPE PERMEABILITY ON WAVE RUN-UP ON A 1 ON 1½ SLOPE
(For isolines of H'_0/T^2)
-----From smooth slope data
0.40
0.25
0.10
0.06
0.02
0.005
R^2/K
10^3
10^4
10^5
10^6
10^7
10^8
10^9
10^{10}
10^{11}
LEGEND
Symbol H_0/T^2
0 -0.012
0.012 -0.04
0.04 -0.08
0.08 -0.17
0.17 -0.32
0.32 -0.70
$\frac{H'_0 T^2}{K}$

FIGURE 21.

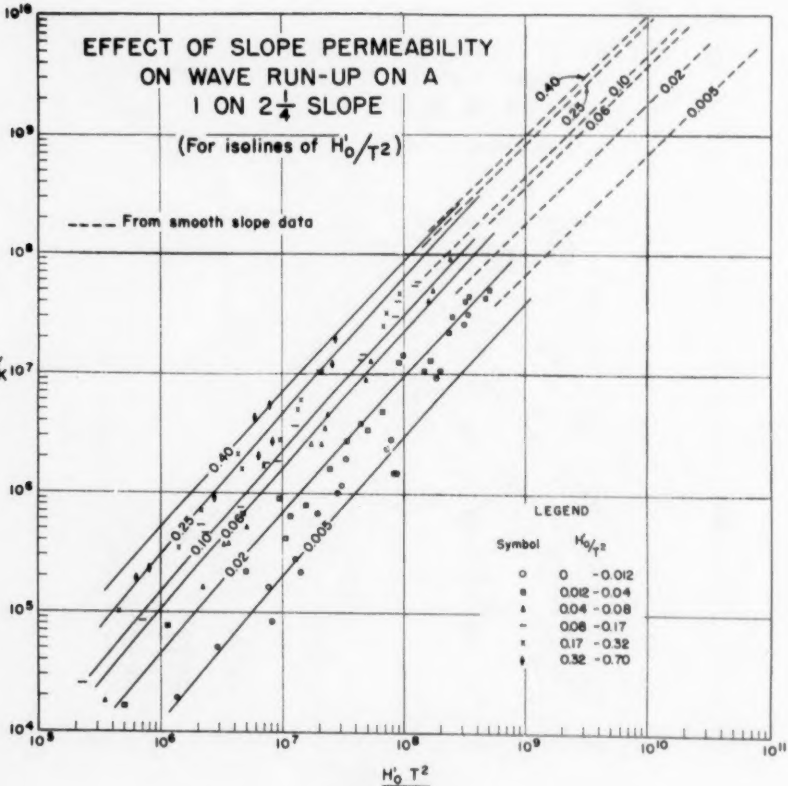

EFFECT OF SLOPE PERMEABILITY
ON WAVE RUN-UP ON A
1 ON 2 1/4 SLOPE
(For isolines of H'0/T2)
---- From smooth slope data
R²/K
H'0 T²/K
LEGEND
Symbol H0/T2
0 -0.012
0.012 - 0.04
0.04 - 0.08
0.08 - 0.17
0.17 - 0.32
0.32 - 0.70

FIGURE 22.

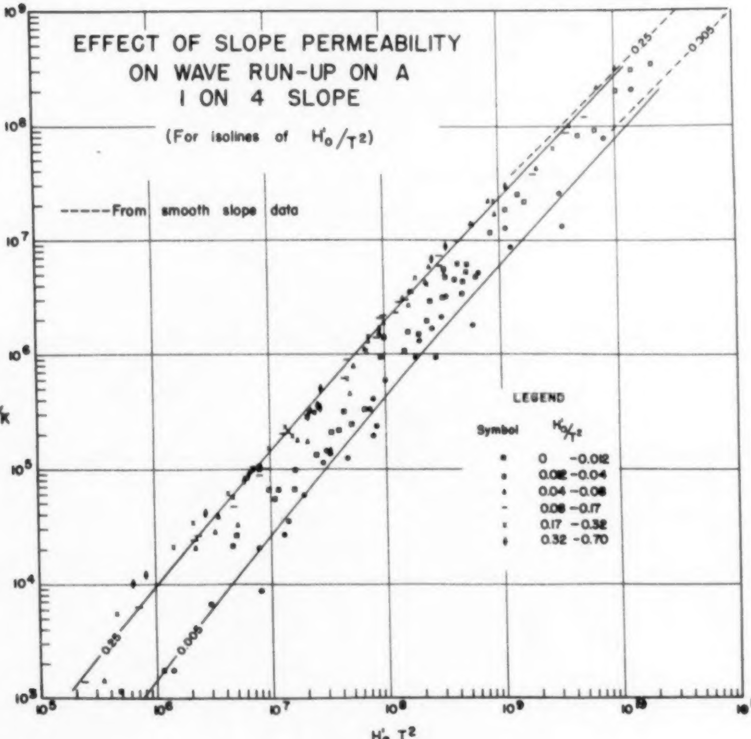

EFFECT OF SLOPE PERMEABILITY
ON WAVE RUN-UP ON A
1 ON 4 SLOPE
(For isolines of H'_0/T^2)
-----From smooth slope data
LEGEND
Symbol H'_0/T^2
0 -0.012
0.012 -0.04
0.04 -0.08
0.08 -0.17
0.17 -0.32
0.32 -0.70
R^2/K
$\frac{H'_0 T^2}{K}$
0.25
0.005

FIGURE 23.

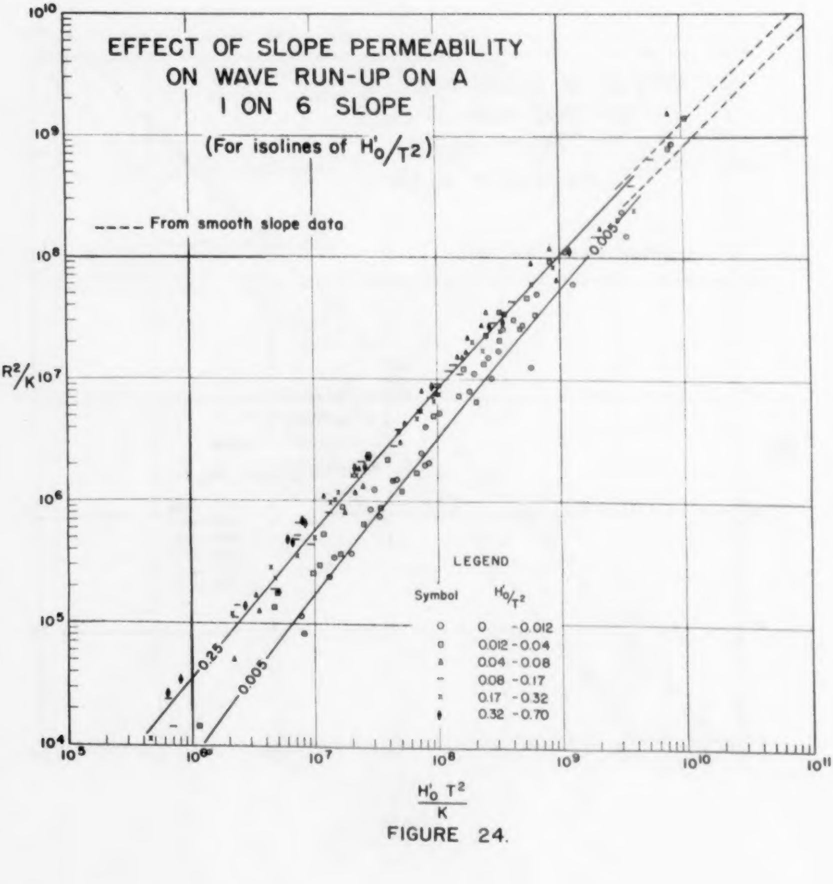

EFFECT OF SLOPE PERMEABILITY
ON WAVE RUN-UP ON A
1 ON 6 SLOPE
(For isolines of H'o/T2)
From smooth slope data
R2/K
H'o T2/K
0.25
0.005
LEGEND
Symbol H'o/T2
0 -0.012
0.012-0.04
0.04 -0.08
0.08 -0.17
0.17 -0.32
0.32 -0.70

FIGURE 24.

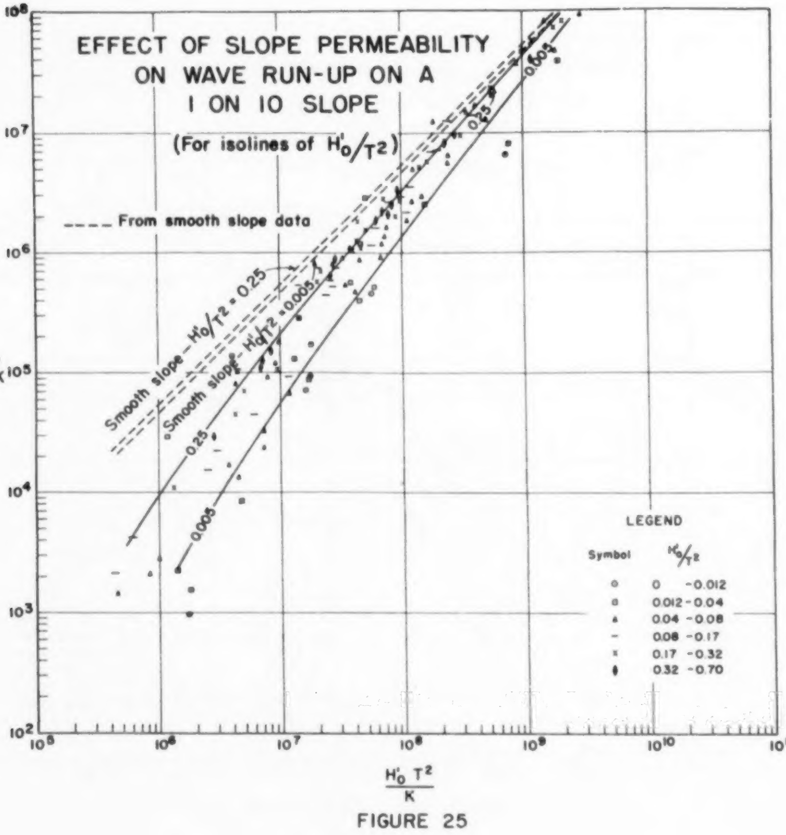
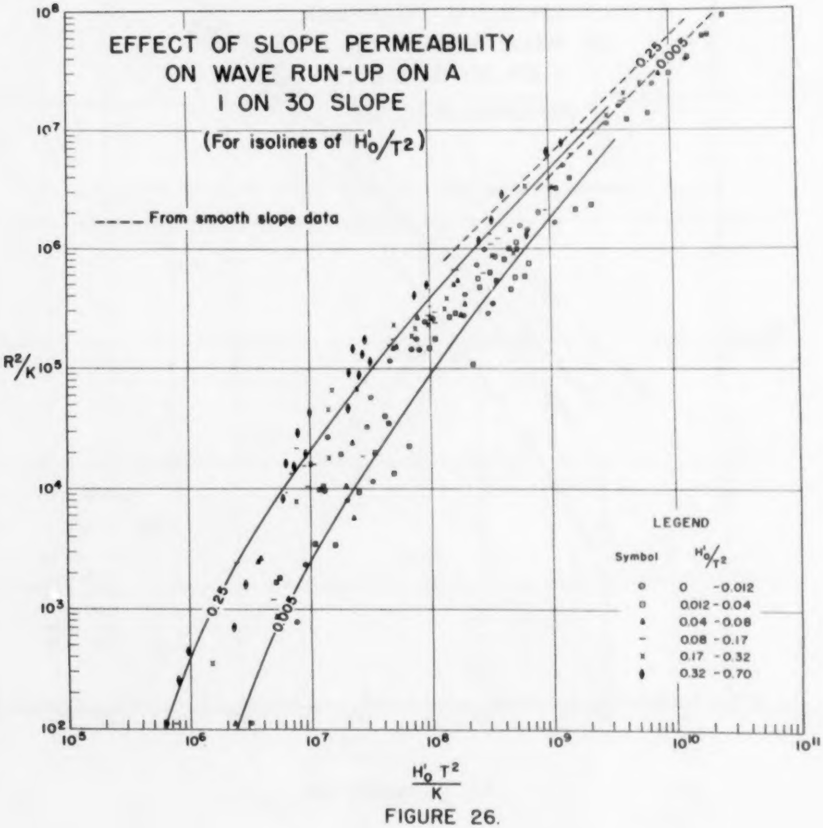
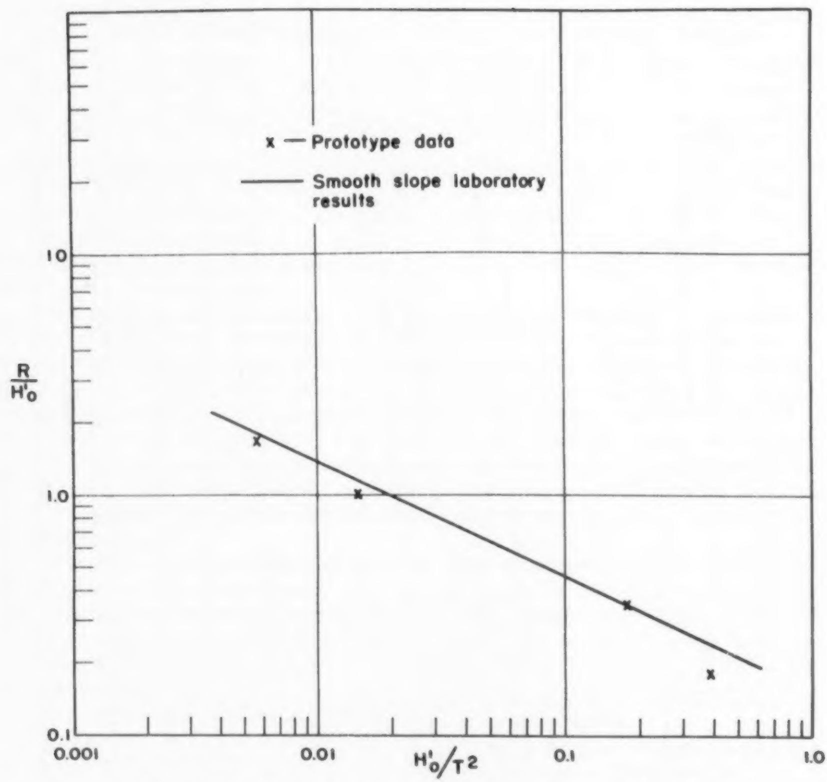

EFFECT OF SLOPE PERMEABILITY
ON WAVE RUN-UP ON A
1 ON 10 SLOPE
(For isolines of H'_0/T^2)
From smooth slope data
Smooth slope - H'_0/T^2 = 0.25
Smooth slope - H'_0/T^2 = 0.005
0.25
0.005
R^2/K
$\frac{H'_0 T^2}{K}$
LEGEND
Symbol
H'_0/T^2
0 -0.012
0.012-0.04
0.04 -0.08
0.08 -0.17
0.17 -0.32
0.32 -0.70

FIGURE 25



EFFECT OF SLOPE PERMEABILITY ON WAVE RUN-UP ON A 1 ON 30 SLOPE
(For isolines of H_0'/T^2)
---- From smooth slope data
0.25
0.005
R^2/K
10^2
10^3
10^4
10^5
10^6
10^7
10^8
10^9
10^{10}
10^{11}
$\frac{H_0' T^2}{K}$
LEGEND
Symbol
H_0'/T^2
0 -0.012
0.012 - 0.04
0.04 - 0.08
0.08 - 0.17
0.17 - 0.32
0.32 - 0.70

FIGURE 26.



PROTOTYPE DATA COMPARED WITH LABORATORY RESULTS (1 ON 15 SLOPE)

FIGURE 27.

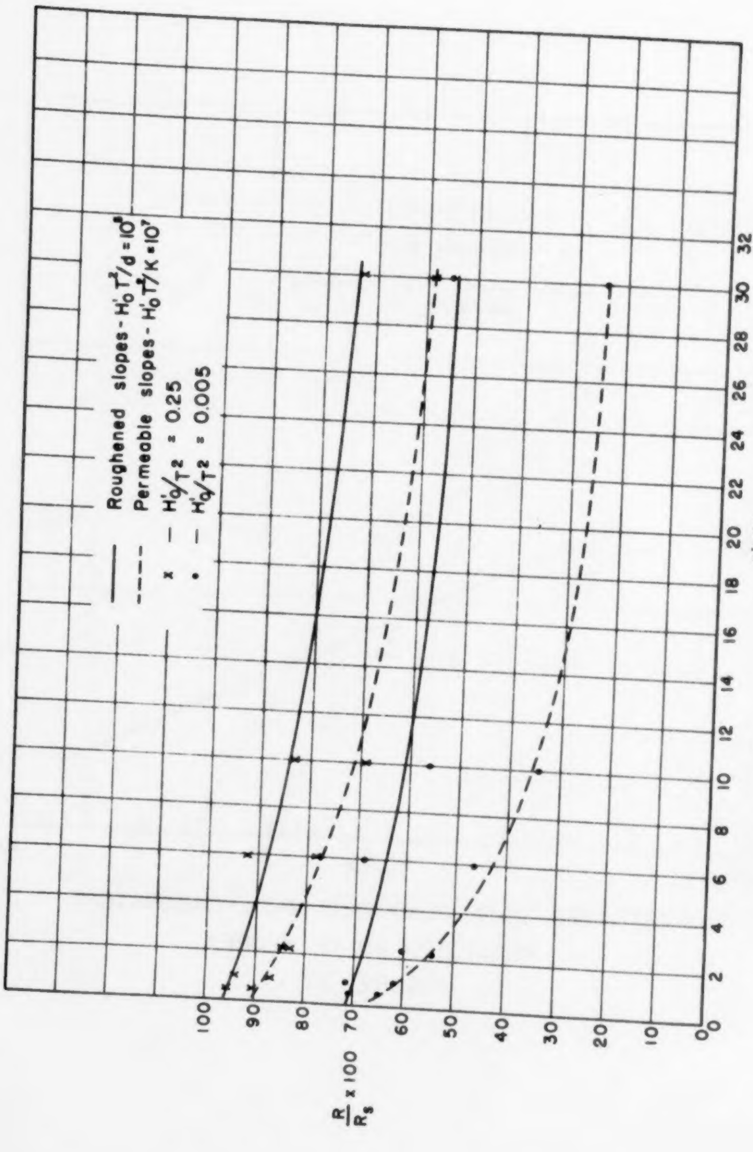


FIGURE 28. RUN-UP ON ROUGHENED AND PERMEABLE SLOPES COMPARED WITH SMOOTH SLOPES

Summary and Discussion

The results of the laboratory experiments on smooth structures of constant slope show that the relative wave run-up is a function of the deep water wave steepness and the structure slope and that, for a constant structure slope, the highest relative run-up for steep waves occurs on a slope in the order of 1 on 2 and the highest relative run-up for waves of low steepness occurs on a slope in the order of 1 on 5.

The results of the experiments on roughened and permeable structures show that the run-up is a function of the deep water wave steepness, the structure slope, and the median diameter of the roughness material for rougnened slopes, or the permeability of the slope material for permeable slopes. For the roughened slopes, the run-up divided by the median diameter of the roughness material is related to an inverted roughness coefficient H'_0T^2/d^2 for isolines of H'_0/T^2. For the permeable slopes, the run-up squared is related to an inverted permeability coefficient H'_0T^2/K for isolines of H'_0/T^2. In both the roughness and permeability tests, the effect of the slope roughness or permeability on wave run-up increases as H'_0T^2/d^2 or H'_0T^2/K decreases and as the slope becomes flatter.

The scatter of the data about the various curves is large for the steeper smooth and relatively smooth slopes and for all of the slope conditions involving the use of the large roughness and permeability materials. However, it is believed that the methods used in correlating the run-up with the various parameters involved show promise and that, in the future, efforts should be made to increase the accuracy of the data by testing larger waves with longer periods. It is hoped that such tests may be made with nearly prototype waves at the Board in the near future. Meanwhile, the user should consider the data scatter in the vicinity of the curves which he uses when applying the results of these tests.

The results of these tests are not valid when the depth of water at the toe of the slope is less than about 3 wave heights since the relative run-up is affected by the depth at the toe of the slope.[2] In general, as the depth at the toe of the slope decreases below 3 wave heights, the relative run-up increases to a maximum value which may be twice the relative run-up for a large water depth at the toe of the slope. From this maximum, the relative run-up decreases as the depth at the toe of the slope decreases further.

The results of the run-up tests on smooth and roughened slopes should be applicable to prototype conditions when the dimensionless parameters involved are within the range of the dimensionless parameters given in the graphs, with the possible exception of conditions where the diameter of the roughness material equals or exceeds the impinging wave heights. Under such conditions, the validity of the curves would be doubtful since this range of wave heights compared to material diameters was not tested. The curves should, however, be applicable to many cases involving riprap slope protection, where only one or two layers of riprap are used.

The results of the run-up tests on permeable slopes should be applicable to prototype conditions in the same dimensionless parameter range. This range should cover all natural beach materials and the test results should be valid for the design of artificial beach fills. Any attempt to use the results of these tests to cover the effect of the "permeability" or porosity of rubble mound jetties or sea walls should be made with the realization that the term "permeability" as used to describe the results of these tests becomes very difficult to obtain quantitatively when the median diameter of the permeable

material is larger than the 10.0mm material used in these tests. Also, such structures are rarely thick enough, front to back, to eliminate the effect of structure thickness on wave run-up.

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List of Symbols and Parameters

Symbol	Definition	Unit
d	Median diameter of roughness material	Ft.
g	Acceleration of gravity	Ft./sec.^{2}
H'_o	Deep water wave height	Ft.
K	Permeability of permeable materials	Ft.^{2}
L_o	Deep water wave length	Ft.
R	Vertical height of wave run-up above SWL	Ft.
S_o	Coefficient of sorting	-
S_k	Coefficient of skewness	-
T	Wave period	sec.
R/H'_o	Relative run-up	-
H'_o/L_o	Deep water wave steepness	-
H'_o/T^2	$5.12\, H'_o/L_o$	-
H'_oT^2/d^2	Inverted roughness coefficient	-
H'_oT^2/K	Inverted permeability coefficient	-
θ	Angle between the face of the slope and the horizontal	Degrees

TABLE I

SLOPES AND MATERIALS TESTED

Beach Material Size (mm)	Roughened Slopes 1/30	1/10	1/6	1/4	1/2½	1/1½	1/1	1/½	Vertical	Permeable Slopes 1/30*	1/10	1/6	1/4	1/2½	1/1½	1/1	1/½	Vertical
Smooth	x	x	x	x	x	x	x	x	x	-	-	-	-	-	-	-	-	-
0.2	x	x	-	-	-	-	-	-	-	x	x	x	x	-	-	-	-	-
1.0	x	x	x	x	x	x	x	x	-	x	x	x	x	-	-	-	-	-
2.0	x	x	x	x	x	x	x	x	-	x	x	x	x	x	x	x	x	x
3.5	x	x	x	x	x	x	x	x	-	x	x	x	x	x	x	x	x	x
10.0	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x

* 5" layer of beach material (except 0.2 mm and 10.0 mm)

TABLE 2

WAVE PERIODS AND HEIGHTS AT THE TOE

OF

THE 1 ON 30 SLOPE INCLUDING CORRESPONDING VALUES OF H_o'/T^2

T	0.72	1.00	1.55	1.85	2.63	3.65	4.70
	0.07	0.05	0.03	0.05	0.03	0.10	0.08
	0.17	0.10	0.13	0.21	0.07	0.21	0.18
H	0.21	0.20	0.27	0.39	0.24	0.33	0.30
	0.26	0.30	0.42	0.46	0.44	0.46	0.45
		0.40	0.56	0.59	0.61	0.60	0.60
	0.14	0.054	0.013	0.015	0.004	0.006	0.0026
	0.33	0.108	0.058	0.064	0.009	0.013	0.0057
H_o'/T^2	0.41	0.215	0.121	0.118	0.032	0.020	0.0097
	0.51	0.322	0.188	0.139	0.058	0.028	0.016
		0.41	0.250	0.178	0.081	0.036	0.019

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RAILROAD BRIDGE ALTERATIONS, CALUMET-SAG PROJECT[a]

George W. Svoboda[1]
(Proc. Paper 1641)

ABSTRACT

The widening of the Calumet-Sag Project waterway requires the alteration of numerous railroad channel bridges whose expense is shared by the bridge owners and the Federal Government. Problems from the standpoint of cost apportionment, planning, and the negotiation of construction agreements with the bridge owners are detailed and complex.

INTRODUCTION

At present (1958) the Illinois Waterway links the Mississippi River and its tributaries to the Great Lakes in an inadequate manner at Chicago. To provide a modern main connection, the Congress authorized the improvement of the Calumet-Sag portion of the Illinois Waterway in 1946 and its construction was started in the latter part of 1955. The improvement will permit large modern barge tows to serve the great industrial areas of South Chicago, Whiting, East Chicago, and Gary. In addition, barge traffic in volume will be able to reach the deep draft terminal at Lake Calumet and also Indiana Harbor where large scale barge-to-lake and lake-to-barge transfers will be possible. Barge-to-rail and rail-to-barge exchange of cargo will also be enhanced upon completion of the improved waterway. That part of the improvement to be located in Illinois is scheduled to be substantially completed in 1963 and the remainder of the project some years later.

The Calumet-Sag navigation project provides for the Corps of Engineers to (1) widen the waterway, where necessary, from Lockport, Illinois to the head of deep draft navigation at Lake Calumet, (2) construct a branch channel

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a. Presented at the Chicago Convention of the American Society of Civil Engineers, February, 1958.

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to connect with both the deep draft portion of the Indiana Harbor Canal at East Chicago and a new barge terminal at Gary to be constructed by others, (3) require the alteration, removal or reconstruction of numerous railroad and highway bridges crossing the project waterway which have substandard vertical and horizontal clearances and (4) construct two new locks and controlling works for the purposes of controlling diversion of water from Lake Michigan and maintaining water levels in the channels landward of the locks at -2.0 feet, Chicago City datum, thus permitting about a 5-foot lesser elevation of low steel of bridges. Existing channels will be either widened from the present narrow usable minimum width of 60 feet or newly constructed, if not now navigable, to a minimum usable width of 225 feet except that portion of the branch channel between the Indiana Harbor Canal and Gary which will be constructed to a width of 160 feet. A usable channel depth of 9 feet will be provided in the improved portions of all channels.

Under the legislation authorizing the project, local interests, i.e. interests other than the United States, are required at their expense to (1) furnish all necessary channel right-of-way and spoil disposal areas, except at railroad bridge crossings, (2) alter or relocate obstructive utilities crossing the channels to be improved, (3) contribute to the cost of reconstructing all restrictive railroad bridges, and (4) remove or alter all restrictive highway bridges. The latter requirement, however, may be changed to provide for Federal participation in cost if legislation to be proposed to the Congress in 1958 is enacted. All other costs are items of Federal expense.

Channel railroad bridge changes surpass by far all other salient project features in scope, difficulty, and cost. Of the total estimated project cost of $195,300,000 on the basis of 1957 prices, railroad bridge relocations will require a total estimated expenditure of $75,930,000 or about 39 percent of the total project cost. One or more railroad bridge structures at each of twenty-four channel crossings, involving sixteen railroads, are scheduled to be replaced. The purpose of this paper is to indicate in a general manner the overall railroad bridge replacement problems and procedures that relate to the apportionment of cost, planning, and the negotiation of relocation (construction) agreements with the railroads concerned.

Apportionment of Cost

The Act of 21 June 1940, commonly referred to as the Truman-Hobbs Act, established a policy for Federal participation in the reconstruction cost of railroad bridges which are determined by the Secretary of the Army to be unreasonably obstructive to the free navigation on any navigable waters of the United States. Prior to the enactment of the Calumet-Sag project legislation, it was considered that the apportionment of project railroad bridge alteration costs between the United States and the owners should be determined either by the procedures established under the aforementioned Act of 1940 or by a more specific method to be negotiated with the owners. After a detailed study of the matter, it was proposed that cost apportionments should be in conformity with the general principles contained in section 6 of the 1940 Act as expanded to a more specific and workable form by an appropriate agreement with the railroads. Such an agreement was executed by the United States and the railroads on February 23, 1945 and its use was authorized by the Congress. The salient features of the agreement are hereinafter explained. Any reference to a

bridge alteration, relocation or reconstruction usually includes the modification of its approaches and operating structures appurtenant thereto.

The true costs of bridge changes and exact allocations thereof are determined at the time construction is completed and are based on formulae predicated upon the following general principles:

a) That the new bridges will belong to the railroads upon completion at which time the Government's interest will terminate after final settlement of cost apportionment.

b) That the railroads themselves will construct and/or contract for the new work when sufficient funds are available to meet the cost of the Government's share of the work.

c) That the total cost of construction will be reduced by the value of salvaged material.

d) That partial payments without interest will be made to the railroads by the Government during the construction period.

e) That the increased cost of railroad operation during construction will be a part of the cost of the work and will not be prorated.

In general, the entire costs of railroad bridge changes are borne by the Government except that each bridge owner bears such part of the cost as is attributable to the (1) replacement of the expired service life of the present bridge, (2) provision of carrying capacity and trackage in excess of that of the present bridge and (3) features which reduce the owner's operating and maintenance costs. In the case of any existing temporary bridges, costs attributable to the owners, as outlined above, are determined on the basis of costs which would have been necessary had permanent bridges similar to those approved for nearby locations been constructed at the time the temporary bridges were originally built and not upon the cost of the existing temporary bridges.

New bridges are separated into the classes shown below with the costs divided on the basis indicated for each group:

Type I. Fixed bridge, with the new channel span longer than the entire existing bridge. The Government and the owner share the cost of the new channel span, consisting of the truss span and the two main piers. The remainder of the bridge is considered as an approach span, for which the Government bears the entire cost.

Type II. Fixed bridge, with the new channel span shorter than the entire existing bridge. The Government and the owner share the cost of the new channel span; also the Government and the owner share the cost of a portion of the new approach spans adjacent to the channel span, so that the cost is shared on a total length of bridge equal to the length of the existing bridge. The Government bears the entire cost of the remainder of the approach spans.

Type III. Lift bridge, with the new channel span longer than the entire existing bridge, and the approach girder spans beyond the tower span. The Government and the owner share the cost of the new channel span and the two tower spans. The Government bears the entire cost of the new approach spans outside the tower spans.

Type IV. Lift bridge, with the new channel span longer than the entire

existing bridge and no approach girder spans beyond the tower span. The Government and the owner share the cost of the entire new bridge.

Type V. Lift bridge, with the new channel span shorter than the entire existing bridge, the entire new bridge longer than the entire existing bridge, and no approach girder spans beyond the tower span. The Government and the owner share the cost of the entire new bridge.

Type VI. Lift bridge, with the entire new bridge shorter than the entire existing bridge. The Government and the owner share the cost of the entire new bridge.

Type VII. Lift bridge, with the new channel span shorter than the entire existing bridge, the entire new bridge longer than the entire existing bridge, and the approach girder spans beyond the tower spans. The Government and the owner to share the cost of the new channel span and the two tower spans. The Government bears the entire cost of the new approach spans outside the tower spans.

A straight-line method of computing accrued depreciation is used in determining the used service life of each existing channel bridge after its actual capital cost less salvage has been determined. Actual capital cost equals the original cost plus additions, minus retirements and plus the net cost of betterments excluding labor. If the original cost is unknown, the cost of reproducing a new bridge at the date of basic valuation by the Interstate Commerce is determined in lieu thereof. The useful life of bridges or major parts thereof are determined in accordance with an approved schedule stated in the apportionment agreement. As an example all steel-bridge substructures are taken to have a useful life of 100 years, with main and branch line steel-bridge superstructures at 70 and 100 years respectively. The cost of removing each existing bridge is shared by the owner and the United States with the owner's share being equal to the ratio which the used service life of the existing bridge bears to its estimated total useful life.

Each owner bears that part of the new bridge cost, if any, due to the increased capacity and trackage on the existing bridge length and the Government absorbs the cost of the existing designed capacity on the increase in bridge length. The increased costs of additional capacity and additional trackage, if any, on the increase in bridge length is prorated between the United States and the owner as shown graphically by figure 1.

Rectangle 1 of figure 1 equals the cost of replacing present capacity on the present bridge layout which cost is borne by the Government after deductions for expired service life.

Rectangle 2 equals the cost of present capacity on the increase in bridge length required for the proposed initial navigation channel. The United States also bears this expense.

Rectangle 3 equals the cost of extra capacity and trackage desired by the owner on the present bridge layout. This cost is borne by the owner.

Rectangle 4 equals the cost of extra capacity and trackage desired by the owner on the increase in bridge length required for the proposed initial navigation channel. The owner bears a share of this cost as determined from the ratio which the used service life of the existing bridge bears to the estimated total useful life.

Rectangle 5 equals the cost of extra capacity and trackage desired by the owner on the increase in bridge length required for that portion of the ultimate navigation channel which lies beyond the width of the proposed initial navigation channel. The owner bears a share of this cost equal to one-half the ratio which the used service life of the existing bridge bears to the estimated total useful life.

Rectangle 6 equals the cost of present capacity on the increase in bridge length required for that portion of the ultimate navigation channel which lies beyond the width of the proposed initial navigation channel. This expense is borne entirely by the United States.

Rectangle 1 to 6, inclusive, apply only to that portion of the bridge on which costs are to be shared as determined by one of the bridge types or classes previously indicated herein. Since the channel widths as authorized by the enabling legislation are also the ultimate widths, Rectangles 5 and 6 are inoperative in the determination of project cost apportionments.

Rectangle C equals the cost of that part of the bridge on which the Government bears the entire cost.

Should fixed bridges replace any movable bridges currently operated, the railroads are required to pay a negotiated amount to the United States in consideration of the estimated savings in operating and maintenance costs. Where fixed bridges replace either movable bridges not currently operated or those which have temporarily been authorized by the Government to remain fixed, the railroads are required to contribute a negotiated amount to the United States in consideration of their being relieved of contingent liability for operating and maintenance costs.

The United States acquires, at its expense, all of the right-of-way and other land necessary for the reconstruction of each railroad bridge and facilities appurtenant thereto; it also arranges, if necessary, for the relocation of any roads and utilities occupying such lands. Upon completion of the relocation work, the acquired right-of-way and other land is conveyed to the railroad by quit claim deed.

If the operations of a railroad are discontinued over portions of its existing right-of-way upon completion of the relocated line, such lands are conveyed by the railroad to the Government by quit claim deed except as otherwise stated below. In the event a railroad desires to retain its right-of-way or any part thereof over which operations are abandoned, credit for the fair market value of the retained lands is given to the United States. In some cases where reversionary rights may be involved, perpetual easements are granted to the Government by the railroad, in liew of quit claim deeds, for abandoned right-of-way required by the United States for channel widening and navigation purposes.

Any lands required because of railroad betterments or for their own convenience are paid for by the railroads. The United States pays the railroads for any of their land required for project purposes as may be agreed. Such payments must not exceed the current valuation placed upon the railroad land by the Interstate Commerce Commission.

With regard to any track changes above the top of the track subgrade, the net cost is determined after crediting the salvage value of the existing track (including rail, tie plates, switches, switch gear, frogs, and miscellaneous parts not used in the new track, but excluding cross-ties) against the total cost

of removal and reconstruction. Of the net cost of track changes, the owner contributes 15 percent of the cost and the United States 85 percent. These percentages were empirically determined and are believed, on an average basis, to be equitable to both parties. They are based on the premise that existing trackage to be relocated will, in most instances, be replaced with trackage utilizing new materials. Thus the owner will benefit to the extent that the expired service life of the existing trackage will be replaced in the new and each owner's 15 percent contribution represents the value therefor. All other track alteration costs, including subgrades except for betterments or additional trackage, are borne by the Government.

Costs of signal and interlocking systems involved in the bridge changes are included as a part of the project. Each owner bears that portion of the present signal and interlocking systems investment costs as the used service life of the systems bear to their total estimated service life. The owner also bears a pro rata share of the removal costs of the present systems on the same basis. Salvage is credited against the total costs. Except for betterments, all other costs are borne by the United States. The useful lives of the facilities are established in accordance with the usual practices of the Interstate Commerce Commission.

When the scope of the work includes structures of a special nature not specifically mentioned in the apportionment agreement, the case is settled with each railroad in accordance with the established general principles governing the allocation of costs. Any features added to fixed bridge structures over the channels to provide for possible future conversion to lift bridges in the event of a national emergency are installed at the expense of the United States. The construction or alteration of grade separations, retaining walls culverts, and other miscellaneous structures as are occasioned by the new work is accomplished at the expense of the Government.

The cost of any changes to power, signal or communication lines necessitated by the project is borne by the United States except that portion of the cost attributable to betterments or other specific benefits occurring to each railroad, if any.

Credit to the railroads for any increased cost of maintenance, repairs, operations or taxes resulting from the relocation of the bridges is not allowed by the enabling Act. Further, each owner is required to waive any claim to a credit for the cost of future removal and renewal of each new bridge at the end of its useful life. The United States, in turn, waives any interest in and claim to a credit for the salvage value of the new bridges. Also any credit for benefits to the railroads arising from the greater efficiency with which railroad traffic may be handled across any fixed bridges replacing movable bridges cannot be claimed by the United States.

Planning

It is the policy of the United States to bear the entire expense of project planning and design. Unless unusual circumstances warrant, necessary exploratory, planning and engineering work is accomplished by each individual railroad under a negotiated fixed-price, lump-sum contract with the United States for engineering services. Due to the complexity, interrelation and joint nature of the railroad relocation work to be accomplished at Blue Island, Illinois involving four railroads, the Government assumed design

responsibility with the concurrence of the railroads. In that case the entire design was performed by a consulting engineer firm under a negotiated contract with the Government. In another isolated instance, the Government produced the necessary designs with its own forces, since the railroad's engineering organization was not available for assignment to such work at that time.

The general engineering requirements are stated by the Government in each design contract as a guide for determining the extent of services to be performed. The design contractor is first required to prepare and submit to the Government for its approval all necessary preliminary layout drawings, design criteria, specifications, schedule of construction, and estimates including the apportionment of costs. After all preliminary engineering work has been approved by the railroad and the United States, the design contractor prepares all necessary final designs, design analyses, plans, specifications, cost estimates including the apportionment of costs and schedule of construction. In connection with the work, the Government makes at its expense all surveys and subsurface explorations that may be necessary when requested by the design contractor. The design contractor coordinates the development of plans with all outside interests concerned, if any.

It is the usual practice of each railroad under contract to perform engineering services to subcontract the design of channel bridges and other major structures to a consulting engineering firm of its selection. The design of trackwork, signals, interlocking systems, and communication facilities is most always accomplished by the railroad. The design contract time specified is usually from one to two years.

All railroad bridges to be reconstructed are designed to provide horizontal and vertical clearances commensurate with the proposed channel widths and type of navigation anticipated to use the improved channels. Minimum horizontal bridge clearances normal to the centerline of the widened channels will be 250 feet for 300-foot channels, 200 and 225 feet for 225-foot channels, and 160 feet for 160-foot channels. Minimum vertical clearance of 25 feet above water surface will be required for all fixed bridges and for all movable bridges in open position. In channels landward of the proposed locks and control works where water surfaces will be maintained at -2.0 feet Chicago City datum the minimum vertical bridge clearance will be +23.0 feet, Chicago City datum, and in the channels lakeward of the proposed locks and control works where the water surfaces will fluctuate with the levels of Lake Michigan the minimum vertical bridge clearance will be +28.0 feet, Chicago City datum. Experience during World War II has indicated that it is practicable to handle both naval and Maritime Commission vessels built along the Great Lakes through the Illinois Waterway and the Mississippi River to the sea. In view of this all fixed bridges are also designed to permit possible conversion to lift bridges in the future affording an additional 15 feet of vertical clearance.

Relocation (Construction) Agreements

After the plans and specifications for the bridge changes are either completed or well-advanced, action is taken by the Government to negotiate a construction contract with each railroad for the relocation, rearrangement or alteration of its facilities. Such contracts are of a cost reimbursable type and are generally known as railroad relocation agreements. Since the facilities

to be constructed will be owned and operated by the railroads after completion, this procedure is believed to be in the best interests of both the railroads and the Government. Also, railroad control of construction is desirable from the standpoint of the movement of railroad traffic in a safe and efficient manner during the construction period.

The scope of the work which may include work of a joint or allied nature involving the property or interests of one or more other railroads is stated in the relocation agreement in as sufficient detail as may be necessary. Each agreement provides that the railroad shall take all action necessary to accomplish the work by means of its own forces and/or through the award of construction subcontracts in accordance with the designs, plans, specifications, and construction schedules which have been approved by the United States, the railroad and other parties concerned, if any.

An appropriate article covering right-of-way is included in each agreement setting forth the obligations and rights of both parties with respect to lands to be acquired or abandoned, rights-of-entries, easements, relocation of roads and utilities, and conveyances. This article also requires each railroad to obtain all permits and approvals as may be necessary from regulatory bodies having jurisdiction such as Federal and State commerce commissions.

Any bids solicited by the railroads for the accomplishment of construction are invited on a competitive lump-sum or unit price basis in a manner as the Government may approve. All bids received are submitted to the United States by the railroad with a recommendation as to award. The Government may request rejection of bids received and require the solicitation of new bids or may approve the award. Any subcontracts that are awarded are executed in such form as the Government nay approve and are supported by whatever bonds the United States and the railroad may prescribe. The usual contractural practices of the railroads are followed in so far as is possible.

With regard to payment, each agreement provides that the Government will reimburse the railroad for its proportionate share of the contract costs in accordance with the provisions of the enabling legislation. The exact amount to be reimbursed to the railroad cannot be ascertained until all work has been completed and the actual costs have been finally determined. In view of this, provisions are made in the agreement for the Government to periodically reimburse the railroad for a stated estimated percent of its actual costs incurred during the construction period. The estimated percentage is determined by applying the formulae for cost apportionment set forth in the Act of Congress to the most accurate estimate of construction cost of the work to be undertaken as is available at the time the relocation agreement is negotiated.

After the work has been completed and when the actual costs of reconstruction have been finally determined, a final apportionment of costs in accordance with the enabling legislation is made. By a modification to the agreement at that time, the amount to be reimbursed to the railroad by the Government is adjusted accordingly and previous under or over payment corrected.

Each agreement also provides that the railroad shall submit to the Government, after the evaluation of bids, a written guaranty that the total cost of the work will not exceed the sum stated in the guaranty. It is further provided that total payments by the United States will not exceed its proportionate share of the guaranteed cost unless the guaranteed cost is exceeded by

reasons of emergencies, conditions beyond the control of the railroad or unforeseen or undertermined conditions. Under such circumstances, the United States may, after full review of the circumstances, provide for additional payments to help defray such excess cost.

As required by Congressional policy, funds for prosecuting the work are only made available in fiscal year increments sufficient to accomplish such work as can normally be placed during that period of time. At the time each relocation agreement is executed, the sum of money available to the railroad for construction payments until the end of the then current fiscal year is stated in the contract. The agreement further provides that beyond the end of the first fiscal year, the work will be continued with funds to be thereafter appropriated. Procedures are prescribed in the agreement involving the reduction or increase of funds allotted to the railroad and the suspension or termination of the work in the event available funds are exhausted and the Congress has failed to provide additional funds during its regular session as expected.

Except with respect to lump-sum or unit price subcontractor payments, all railroad cost records and supporting data of expenditures for which proportionate reimbursement will be claimed from the Government are required to be available to Government representatives for auditing purposes. Monthly payment bills either for subcontract work or work accomplished by the railroad's own forces are required to be supported in the manner prescribed in the agreement.

Under the terms of each agreement, the railroad may either provide its own construction supervision forces or negotiate a subcontract for such services with a consulting engineering firm. Generally the railroads prefer the latter arrangement due to the unavailability of their normal forces for such purposes.

Based upon the principles of cost apportionment and 1957 cost estimates, it is estimated, on an average basis, that about 86 percent of the total railroad bridge relocation project costs will be borne by the United States.

CONCLUSION

The alteration of railroad bridges crossing the waterways of the Calumet-Sag navigation project has presented many detailed and complex problems from the standpoint of cost apportionment, planning and the negotiation of relocation agreements. These problems are all being met and solved by the procedures which have been devised.

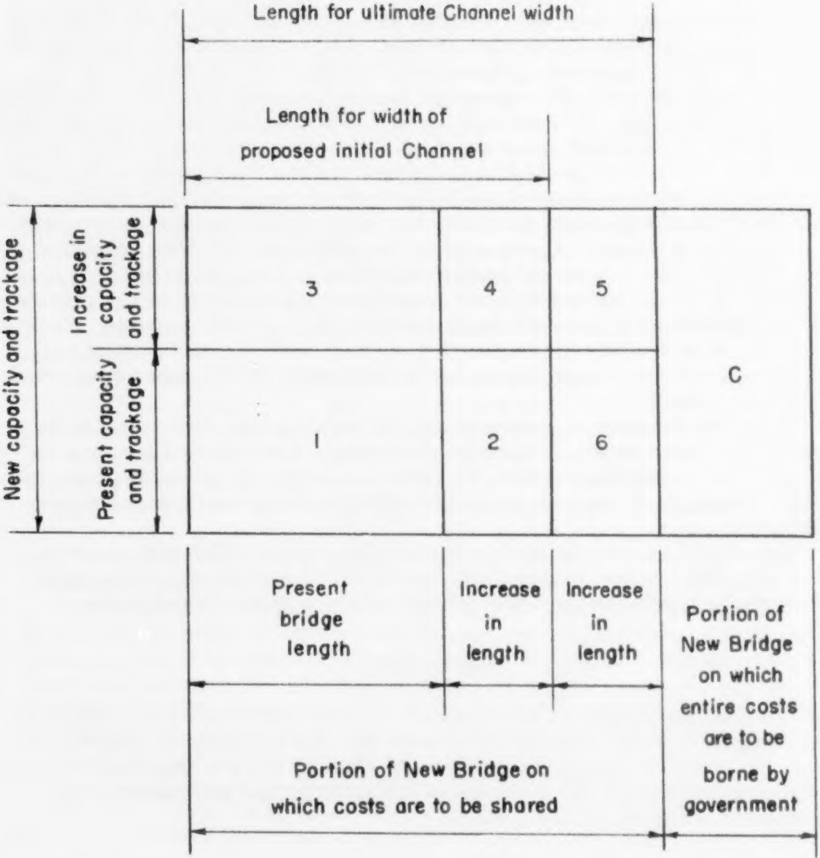

FIG. 1 COST DIAGRAM - ALLOCATION OF BRIDGE COSTS

Journal of the
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CALUMET RIVER LOCK, CALUMET-SAG PROJECT, ILLINOIS[a]

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(Proc. Paper 1642)

ABSTRACT

The new Calumet River Navigation Lock will involve departures from the usual design in that the lock walls and guide walls will be of steel sheet piling construction, with concrete construction being more or less limited to the gate blocks. Lock gates will be of the sector type, with a total span of 110 feet.

INTRODUCTION

For those who are not familiar with navigation locks, a few introductory remarks are perhaps in order. The primary function of a navigation lock is to overcome an abrupt change in water surface elevation in a navigable stream, thus enabling water traffic to safely and efficiently pass from one surface elevation to another. Such abrupt change in water surface elevation in a navigable stream may be natural or manmade, such as a dam. A lock is simply an elevator, shaped long and narrow to accommodate its passengers. The water surface in the lock chamber is the floor of the elevator and the lock gates are the elevator doors. As in elevators the lock gates must be closed before vertical motion is permissible. Vertical motion, i.e., raising and lowering the water surface in the lock chamber is caused by admitting water into the lock chamber from the river upstream of the lock and by discharging

Note: Discussion open until October 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1642 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 3, May, 1958.

a. Presented at the Chicago Convention of the American Society of Civil Engineers, February, 1958.

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water from the lock chamber into the river, downstream of the lock.

This paper is not intended to be a manual for design of navigation locks nor is it meant to be solely a description of the Calumet River Lock soon to be built. An effort has been made to concentrate attention on those items wherein the design departs from the usual or where it is felt some contribution has been made.

General

The Calumet River Lock is an item of Federal construction included in Part I of the Calumet-Sag Project of the Illinois Waterway. Part I of the Calumet-Sag Project, as indicated on Plate I, extends from Sag Junction, the junction of the Calumet-Sag Channel and Chicago Sanitary Ship Canal, through the Calumet-Sag Channel and the Little Calumet and Calumet Rivers to Turning Basin No. 5 on the Calumet River. Turning Basin No. 5 is the head of deep-draft navigation on the Calumet River and is located at the entrance to Lake Calumet where the Chicago Regional Port District has constructed a large water-rail-truck terminal. In addition to lock construction, Part I of the Calumet-Sag Project provides for construction of a channel with a minimum width of 225 feet and a usable navigable depth of 9 feet, and the replacement of twelve (12) railroad and eleven (11) highway bridges which have restrictive clearances. More detailed information regarding these and other items of work in the Calumet-Sag Project is beyond the scope of this paper and consequently has not been included.

Flows in the Calumet-Sag Channel are at present controlled by the existing small and inadequate lock at the eastern terminus of the Calumet-Sag Channel at Blue Island, Illinois. The existing lock at Blue Island, which has a width of 50 feet and a length of 360 feet, will be removed and replaced with the new Calumet River Lock and Control Works to be located just south of Turning Basin No. 5. The locations of the existing Blue Island Lock and the new Calumet River Lock and Control Works are shown on Plate I.

The new Calumet River Lock and Control Works will prevent reversals of flow into Lake Michigan, regulate the amount of diversion from Lake Michigan, and control water levels landward of the lock at -2.0 feet, Chicago City Datum. Lakeward of the lock the water levels will be uncontrolled and fluctuate with the level of Lake Michigan. Inasmuch as bridges for the Calumet-Sag Project must have a minimum vertical clearance of 25 feet above the water surface for unobstructed movement of towboats and barges, the maintenance of water surfaces at -2.0 feet, Chicago City Datum, landward of the lock, will permit construction of bridges with low steel at +23.0 feet, Chicago City Datum. In contrast, without the lock, the water surface would be uncontrolled and fluctuate with the level of Lake Michigan, thus requiring that the low steel for bridges be placed at elevation +28.0 feet, Chicago City Datum, to provide a minimum vertical clearance of 25 feet a majority of the time. The savings in cost of bridge construction in lowering the minimum elevation of low steel 5 feet, i.e., from +28.0 feet to +23.0 feet, Chicago City Datum, more than justifies the cost of the lock.

Inasmuch as the Calumet-Sag Project, including the Calumet River below Turning Basin No. 5 is a part of the Illinois Waterway, only towboats and tows of barges will use the lock. Deep-draft vessels, such as lake and ocean ships, will not be able to reach or use the lock.

With the channel of the Calumet-Sag Project designed to handle barge tows consisting of 8 barges, two abreast, plus the towboat, it is basic that the lock be sized also to accommodate this grouping without rearrangement of tows or double lockages. As the standard barge size is 35 feet by 195 feet and the towboat may be about 150 feet long, the overall length of standard barge tow is about 930 feet. During the lock filling operation the water in the upstream portion of the lock chamber may, at times, be sufficiently turbulent to prevent the tying up of tows withint 40 to 50 feet of the upstream gates. Thus, on the basis of a 930-foot tow and provision of 50 feet of lock chamber to absorb possible turbulence at the upstream end and 20 feet for clearance between the tow and the downstream gate, a 1000-foot length of lock chamber (from the hinge of the upstream gate to the skin of the downstream gate) was established. As in other locks on the Illinois and Mississippi Rivers, of which this lock is one link in the chain, the lock has a standard width of 110 feet. The sill elevations have been established at -17.0 feet, Chicago City Datum, to allow 14-foot depth of water over the sills during the lowest reasonable lake level of -3.0 feet. Top of the lock wall has been set at elevation +7.0 feet to provide two feet of freeboard above the maximum recorded Lake Michigan level of +5.0 feet.

Plate II shows the overall arrangement of the Lock and Control Works. The lock walls and the control works are cellular steel sheet piling. The chamber sides of the lock walls are capped with a reinforced concrete cantilever. The lock gates are of the sector type, and the gate bays are of reinforced concrete. Guide walls of tied-back steel sheet piling are 1000 feet in length at each end of the lock to accommodate the 930-foot long barge tows. The control section of the control works is reinforced concrete with four vertically operated sluice gates. Access is provided to the west side of the lock and to the east side of the control works by roads connecting to the Calumet Expressway and Torrence Avenue, respectively.

The normal difference in water levels at the lock is 0 to 5 feet, but under extreme conditions this differential may reach 9 feet. A reverse head is also possible and, under certain theoretical conditions, this head may also reach 9 feet.

Lock and Guide Walls

Both the lock walls and the guide walls are steel sheet-piling structures. As in all engineering design, the objective was to provide a structure of adequate life at least overall cost, including maintenance cost. Normally lock walls are concrete gravity structures because of their permanence and low maintenance cost. However, the first costs of concrete gravity walls are high, attributable in large part to the frequent necessity for a cofferdam in conjunction with construction of the wall. This is the principal advantage of the steel sheet piling wall; no cofferdam is needed. The sheet-piling wall can be built for approximately the price of the cofferdam alone; therefore, the first cost is approximately one-half that of the usual construction of concrete gravity walls. However, it must be stated that if culverts or tunnels are to be provided in the lock walls for purposes of filling and emptying the lock chamber, the concrete gravity wall would undoubtedly be the most economical and/or feasible. However, the Calumet River Lock, with sector gates, does not require tunnels or culverts in the lock walls.

It had been generally observed that steel sheet piling used in breakwaters, piers, etc., in the Great Lakes area developed little or no corrosion, even at the water level, of some of the structures which were over 25 years of age. As a check, soil and water samples at the site of the Calumet River Lock were tested for corrosivity. The results of the tests indicated that corrosion of the piling would not be sufficient to warrant the provision of cathodic protection at this time. It is anticipated that the steel piling will prove very durable and that the maintenance cost will be low. As a precaution, however, against insufficient knowledge of changing conditions, inspection piling will be driven at several locations immediately adjacent to the lock. This piling will be electrically bonded to the lock structure, and so arranged that it can be pulled and examined for evidence of corrosion in the future. Then, if found warranted, cathodic protection can be applied.

As described more fully in a later discussion of the gate bays, the lock site is in a glacial till deposit with 10 to 15 feet of mud and clay overburden, and with bedrock 50 to 60 feet below ground surface. In order to determine the depth to which sheet piling could be driven into the till, pile driving tests were performed in the Fall of 1956. Piles were driven in the region of each of the walls. It was found that the piling can be driven to elevation -35.0 to -40.0 feet, i.e., some 20 feet into the till, without great difficulty and that this depth would provide the necessary structural support. In summary, the foundation conditions are favorable for steel sheet piling construction.

Both the river and land walls of the lock are of cellular construction, the river wall being 23 feet wide and the land wall 28 feet wide. The general construction is shown on Plate III. The earth within the cells is to be removed down to top of lock floor, i.e. at elevation -18.5 feet, and the cells are to be completely backfilled with gravel. When the lock chamber is unwatered for repairs or other reasons, the river wall must support the full water load on the river side and the land wall must support the full earth load on the land side.

During the process of design the top 10 feet of the steel cells on the lock chamber face were replaced with a sturdy reinforced concrete cantilever structure to provide a straight surface for the barges to rub against. This "L"-shaped cantilever structure is supported by the cell walls on the lock chamber side and by a row of "H" piles located in the interior of the cells. Incidentally, a comparative cost study disclosed that this concrete structure was more economical than the portion of the cellular wall which it replaced. Thus, with this concrete structure providing a straight and relatively smooth surface for barges to rub against, it is believed that the last objection to cellular type lock walls has been removed. Hereafter, in locations with similar conditions and where corrosion is not a problem, the great economy of these steel sheet piling cellular lock walls should dictate their use in future designs. However, as previously mentioned, for large locks having high heads (with or without reversal), provision of culverts and/or tunnels in the lock walls will probably be required for purposes of filling and emptying, and, considering the fact that such construction will require unwatering of the lock site, the use of steel sheet piling cellular cells instead of concrete walls not be made arbitrarily.

At first thought it might appear that a single row of tied-back steel sheet piling would be the more economical design for the land wall of the lock chamber. However, an investigation revealed that under the condition of unwatered lock chamber a single row tied-back steel sheet piling wall, when

constructed of the heaviest available Z-type piling, would require additional support at the lock floor to adequately support the resultant loads. A comparative cost study was made of the single row tied-back steel sheet piling wall with struts between lock walls at floor level and of the self-supporting cellular design. The cost of the former was found to equal the cost of the latter, and the cellular design was adapted because of its obvious superior features.

The guide walls, not being subject to the large loads resulting from an unwatered condition, are of tied-back sheet piling construction, the anchor wall being continuous sheet piling. No cofferdam is, of course, required for the construction of these walls.

Gate Bays and Sills

A thorough study of subsurface conditions at the lock site indicated that the reinforced concrete gate blocks and sills could each be separate self-supporting structures without unreasonable differential settlement. In this respect, it is to be noted that a monolithic U-frame across a 110-foot lock would have required a very thick reinforced concrete floor slab which would be considerably more costly to construct. Each gate block is shaped to accommodate the gate leaf and gate anchorages, supports for the emergency bulkheads, and the short loop culverts, the latter being required in the upstream blocks only. Internal stresses in the floor of the leaf recess and stability of the block as a whole require a 10-foot thick floor slab. The size and shape of gate blocks are shown on Plate IV.

Subsurface explorations established that the surface material at the site is a very soft mud to a maximum depth of about five feet. Below this soft surface mud there is about a 10-foot layer of soft to medium clay which overlies a hard compact glacial till. The till extends to a depth of 50 to 60 feet below surface where bedrock is found. Laboratory tests of the till soil samples disclosed that the preconsolidation load on the foundation till exceeded the loads that will be placed upon it by the gate bay structure. In other words, the till has been loaded in the past more than it will be with the lock structure placed thereon. The base of the gate blocks is well into the till, being 28 feet below ground surface. A settlement analysis study indicates that it would take approximately one year, under construction condition loading, to produce a half-inch differential settlement and that the ultimate probable differential settlement would be one inch. The maximum foundation pressures during construction will be about 3.0 kips per sq. ft. and, under normal operating conditions, about 2.0 kips per sq. ft.

It is obvious that a differential settlement of even this or somewhat larger magnitude between the gate blocks or blocks and sill could cause serious binding of the lock gate leaves. To prevent this two possible courses of action were considered. The first was to support the gate blocks on piling, thus reducing the amount of possible differential. This was discarded as being undesirable and costly due to the difficulty that would be encountered in driving piling through glacial till with numerous large boulders. The second course of action, which was more desirable and less costly and ultimately adopted, was to make the gate hinges and sill seals adjustable to compensate for the settlement. With these adjustments a differential settlement between gate blocks approaching two inches can be handled. Most of the adjustments are

of the usual detail. The horizontal adjustment of the top hinge, however, is worthy of special comment. In the past this adjustment required a complex and costly assembly. As now designed for the Calumet River Lock, it is very simply accomplished with eccentric bushings, an eminently sturdy detail that makes it possible, after the lock is in operation, to move the hinge pin one inch in any direction from the theoretical center. This permits a three-inch raising or lowering of the skin plate face of the leaf. Plate V shows the top hinge anchorages assembly, with the eccentric bushings.

Sector Gate

The water level in Lake Michigan may vary from 0 to 5 feet above that in the Calumet-Sag Channel. However, under certain conditions, the head differential may be reversed, that is, the channel level would be above lake level. This occurred recently as a result of heavy rainfall on 12 July 1957. While being somewhat on the conservative side, the design head differential has been taken to be 9 feet both ways. This requires a lock gate capable of withstanding and operating against reverse flows. The sector-type lock gate performs this function admirably. To those unfamiliar with sector-type gates, each leaf can be visualized as a giant piece of pie pivoted or hinged about a vertical axis through the narrow end. The wide circular curved end is centered on the hinge, and is covered with a skin plate to provide the damming surface. Loads on the structure are carried by an arrangement of space-framing to the hinge. Plates VI and VII show the composition of a gate leaf. All water loads are normal to the surface of the skin plate and therefore pass through the center of rotation of the gate leaf. Thus, there is no closing or opening moment due to the water load regardless of the direction of pressure.

Sector Gate Support

The gates for the Calumet River Lock are notable since this is the first time, to our knowledge, that sector gates are employed in a lock with a width as great as 110 feet. The radius of each leaf is approximately 61 feet.

The only connection between the leaf and the gate block is through the hinge, which consists of two parts. The top hinge is designed to take reactions in the horizontal plane only and consist essentially of a vertical cylindrical pin. The bottom hinge, or pintle, must then be capable of supporting all the vertical load as well as reactions in the horizontal plane. To accomplish this a hemisphere is provided as being most suitable to take the resultant reaction inclined to the horizontal. It is the dead load on these hinges and on the space framing that makes these sector gates unusual. With a 61-foot radius, with the center of mass near the skin surface, and with the shallow depth of leaf (only 21 feet), the horizontal dead load reactions are more than twice the dead weight of the leaf. To relieve these loads the provision of rollers at the bottom of the leaf near the skin plate naturally comes to mind and was seriously considered. There are several basic arguments against the use of rollers, such as the debris problem and the necessity, with rollers, of designing the top hinge to take large compressive loads instead of large tensile loads, which are more readily supported. The elimination of a large compressive load on the top hinge by the omission of rollers results from the dead load reaction being in opposition to the reaction from the normal hydrostatic water

load on the skin surface of the leaf. There are still other reasons for omission of rollers but the most compelling one is that the rollers are not considered as accomplishing their intended primary objective, that is, the significant reduction of load in the space framing of the leaf and in the hinge. The leaf is a stiff structure, the vertical dead load deflection at the skin (61 feet from the hinge) being only one-half inch. Any malfunction of the rollers resulting from wear, differential settlement of gate bays and gate sill, loss or failure of a roller, would increase the load in the hinge up to, in the latter case, full dead load reactions. While some overstress could be permitted for this extreme condition, the savings made possible do not compensate for the other apparent disadvantage of rollers.

While the Calumet River Lock gates have relatively small differential in head and are comparatively shallow in depth, it is believed that similar gates with greater differentials in head and of greater depth, but without rollers, can be designed for this size lock, or larger if required, without any unusual design problems.

Lock Filling System

It is believed advisable at this time to briefly discuss the reasons why sector-type lock gates were selected instead of the miter-type gates which are prevalent on the Illinois Waterway and the Mississippi River. For most locks, including those with miter-type gates, the filling and emptying of the lock chamber is accomplished through culverts in the lock walls and gate blocks, thus by-passing the lock gates. When the lock chambers are filled and emptied solely through culverts which by-pass the lock gates, lock gates which cannot operate against differentials in heads can be employed. Thus, such gates cannot be used for filling and emptying the lock chamber and must remain either in a fully open or closed position until the filling or emptying operation is completed.

In contrast, sector gates are capable of operating against differentials in head. This permits the sector gates to be used in filling and emptying the lock chamber, thus eliminating the necessity of constructing costly lock walls of the gravity type with culverts. However, the use of sector gates for filling lock chambers may have the disadvantage of creating undesirable surges in the lock chamber which in turn can result in excessive hawser line tensions for tows occupying the chambers during filling operations. Of course, these surges can be reduced by decreasing the rate of opening of the sector gates, but this may extend the time of filling the lock chamber to the extent that any economic advantage gained by using sector gates for filling purposes may be lost.

In order to thoroughly analyze the use of sector gates in the Calumet River Lock and to determine the most efficient method of operation thereof, a model study of the lock was made at the Waterways Experiment Station, Vicksburg, Mississippi. In summary, the model tests indicated that by use of the sector gates and short by-pass culverts in the upstream gate blocks an acceptable minimum lock filling time could be obtained without excessive hawser line stresses for tows occupying the lock chamber. Accordingly, the design of the lock includes the 10-foot square culverts in the upper gate blocks as shown on Plate IV.

As a matter of interest, it might be mentioned that consideration was given

to the use of "Ears" on the sector gates to eliminate the undesirable surge created in the lock chamber during filling operations. The Ears are projections on the sector gate leaves at the side frames which, in conjunction with the shape of the inner walls of the gate bays, permit the entrance of a large volume of water into the lock chamber around the leaves. The theory is that the water flowing around the gate leaves reduces the velocity of the water entering the lock chamber at the center between the opening of the gate leaves, thus decreasing the resulting surge and hawser lines stress. While Ears on sector gates have been used in the past, they have probably never been an entirely satisfactory solution to the problem of establishing the proper relationship between quantities of center and side flows.

Another major objection to the use of Ears on sector gate leaves is the ice problem in the Chicago Area. At an existing lock in the Chicago Area, where the sector gates do not have Ears but are shaped so that large quantities of water are diverted around the gate leaves, considerable difficulty is experienced during the winter months due to the accumulation of ice in the gate bays each time a lockage is made. To maintain the lock in operation it is necessary for a crew of men to pole the ice out of the bays to make it possible to fully open the gate leaves.

In view of the foreseeable difficulty with Ears on gate leaves due to ice conditions together with the fact that model studies indicated that the sector gates with short culverts through the upper gate blocks would give more satisfactory performance, the use of Ears on the gate leaves was given no further consideration. Even without the problem of ice interference, the present design is superior to one with Ears since the portion of flow around the gate can be varied after construction to take advantage of experience gained in operation.

Gate Operating Equipment

A sector gate is readily adaptable for operation with a "rack and pinion" type of drive. This consists of a rack mounted along the entire upper perimeter of the curved portion of the gate leaf, with a pinion gear supported on the wall of the gate block. The operating forces are a minimum with this method of gate operation as the tangential force has a lever arm of 61 feet about the center of rotation of the leaf. However, there is one disadvantage. In a gate leaf of this unusual size the change in length of the leaf due to temperature approaches three-fourths of an inch, which is sufficient to cause improper tooth contact and result in excessive loadings. Because of this difficulty a hydraulically operating strut was considered in lieu of the rack and pinion. With strut operation it would be necessary to connect the strut to the leaf near the hinge to avoid an excessively long strut. Because the lever arm about the hinge would be relatively short, the operating force would become relatively large; i.e. about five times the force required by the rack and pinion. The strut arm and operating machinery would be large and costly and the loads to be carried by the gate space framing and hinge would be considerably greater. In addition, a considerable enlargement of the gate block would be necessary to accommodate the hydraulic cylinder and supports for the strut arm.

Because of the many apparent disadvantages of using a short strut arm, a solution to the temperature problem was sought so that the much simpler and

more economical rack and pinion could be employed. The design ultimately selected maintains the teeth in proper contact at all times, regardless of change in length of the gate leaf, by providing a radial force (i.e. keeping the teeth meshed) proportional to the tangential force. This is accomplished by floating an idler gear between the pinion and the rack, with the gear automatically kept in proper contact with the rack by a hydraulically operated piston. The piston is connected to the main hydraulic pump and is designed in such a manner that the force exerted is always greater than the separating radial force between the gear and rack. Plate No. VIII shows the operating machinery layout. An electric motor and a hydraulic motor and pump supply the necessary operating force through a speed reducer and a system of gears to the idler gear. The hydraulic transmission is capable of providing leaf speeds from practically zero to 30 feet per minute, measured at the rack. In this connection, the present plan of operation provides for opening the gate leaves at a speed of 2 degrees per minute until the water level differential on the gate reaches 6 inches, at which time the leaves are automatically shifted to a speed of 30 feet per minute.

Operating Controls

Each gate is operated with controls housed at both sides of the lock. These controls are always provided at the gates to give the lock operator an on-the-spot check and control of the gate action. The duplication of controls on each side of the lock keeps locking time to a minimum, especially in a lock of this length, by making it unnecessary for the operator to walk twice the length of lock (one-half mile) should he be on the non-control side of the lock with the gate open. Each leaf is independently operated by a hold-type of switch, that is, the switch must be held manually during closing or opening. Once the switch is thrown, however, the phasing with the culvert valve movement and the change in speed of the leaf proceeds automatically. This is accomplished by a system of float switches, limit switches, and interlocking controls. Additional controls at the culvert valve operating machinery make it possible to independently open or close the valve. The sluice gate controls are of the same type; all control being directly at the gates.

Commercial power is used for operation. A diesel-driven generator will furnish the usual standby power.

Lock Bulkheads

It is necessary to provide some means of unwatering the lock, or at least the gate bays, so that maintenance work can be performed on the gates, sills, anchorages, etc. To accomplish this maintenance, lock bulkheads are provided. Vertical recesses are constructed in the sector-gate blocks just above and below each lock gate to support the lock bulkheads which consist of a tier of steel units, one on top of the other—reminiscent of the old wood stop-logs. Placement of the bulkhead units is to be accomplished with floating plant, the units being stored on the land wall just above the upper gate. The upstream bulkhead recess is located upstream of the intake port of the gate by-pass culvert so that the culvert can also be unwatered for inspection. These lock bulkheads are identical to those that were designed several years ago for Lock 27 on the Mississippi River. The identical design has also been used to

provide bulkheads for the locks in the Illinois Waterway, and locks in other Districts of the Corps of Engineers.

With respect to unwatering, it is to be noted that the existing lock at the entrance to the Chicago River has been constructed to permit unwatering of each individual sector-leaf recess which makes possible the performance of necessary maintenance of the gate leaves without unwatering the lock. More important, work can be accomplished on the leaves without interrupting traffic by taking advantage of the periods when the Lake and River water levels are nearly equal, at which time traffic may proceed through the lock without lockage. This principle is being applied to the Calumet River Lock and will be accomplished with a bulkhead across the leaf recess, at the face of the lock wall. The damming surface is a single row of steel sheet piles supported by the concrete floor at the bottom and by a girder spanning the recess at the top. With this bulkhead in place, access to every part of the leaf, except the upstream face of the skin plate, is possible. Even the rubber seals at the perimeter of the damming surface of the leaf are accessible for replacement. It is understood that all maintenance work on the Chicago River Lock gates to date has been accomplished using these leaf-recess bulkheads, thus avoiding any interruption in traffic.

Flow through the culvert by-passing the upstream sector-gate is regulated with a simple vertical slide-gate valve which is operated by a commercially available tandem-stem screw hoist located directly over each gate. It has been possible because of the low head differential at this lock to omit rollers on this valve and thus greatly simplify an otherwise complicated sealing problem. Under maximum head the sliding friction constitutes over 80 per cent of the hoist capacity required, but the forces are well within the capacity of commercially available hoists. To unwater the upstream sector-leaf recess for maintenance work on the leaf, it is necessary that the by-pass culvert be closed. At the same time it is also desirable to be able to inspect and work on the culvert valve. To accomplish this a bulkhead identical to the culvert valve, except that the hoist is omitted, is furnished. This bulkhead is placed in a slot located just upstream of the culvert valve.

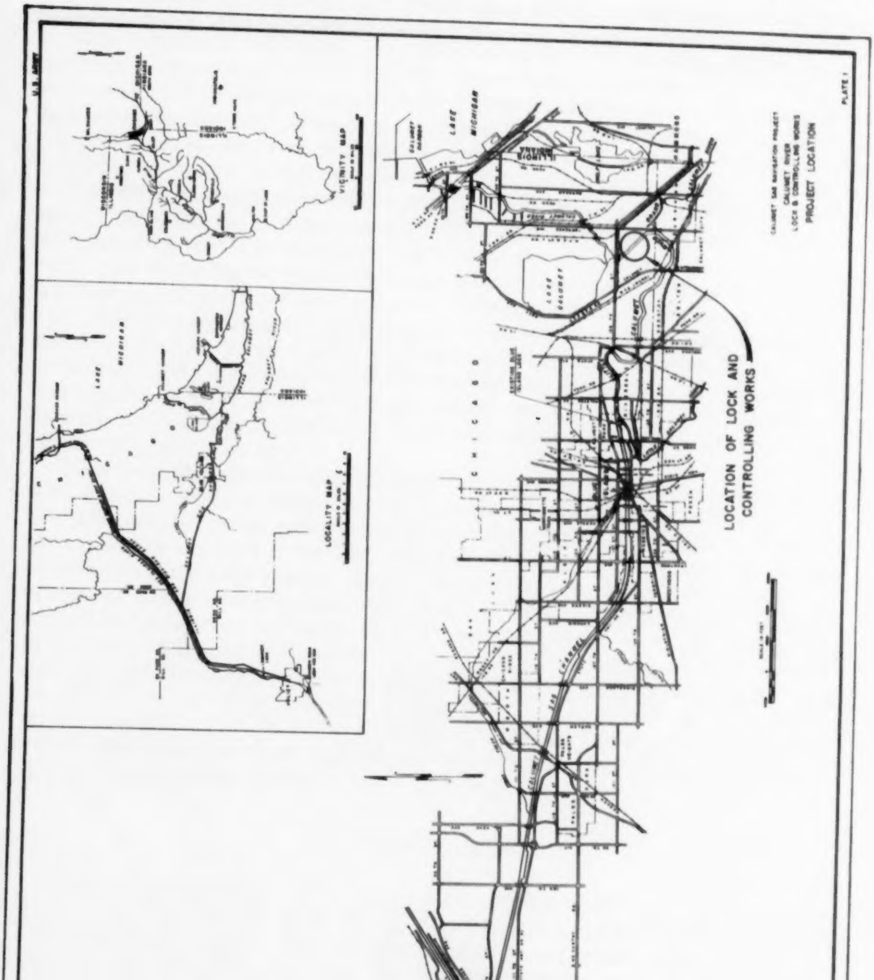
Controlling Works

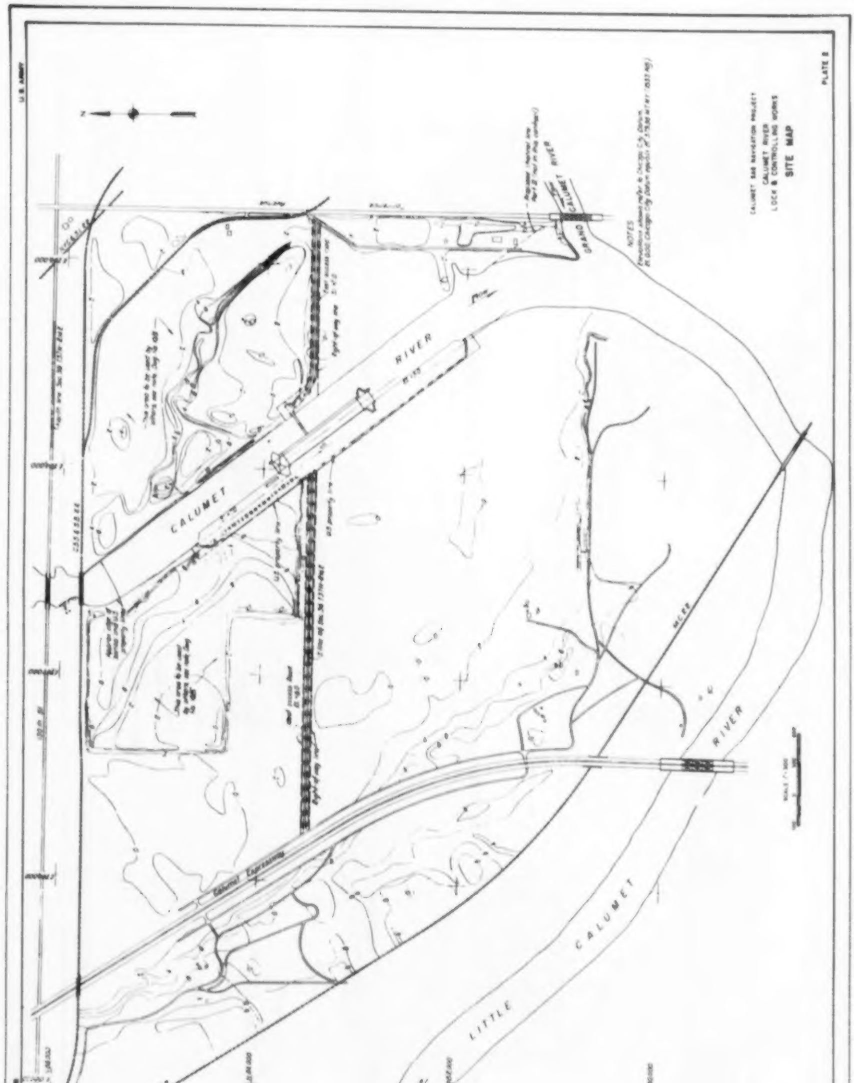
The remaining width of the channel not occupied by the lock is closed by a dam, in the central portion of which a control section is provided. Plates III and IX show the construction planned. The dam, exclusive of the control section, is of cellular steel sheet-pile construction the same as the river lock wall except that the width is somewhat reduced since the overturning moments are less. The moments are less because the river bed, which is the base of the dam, is approximately 3 feet above the lock floor and because the dam is never subject to one side being unwatered as occurs on the lock walls.

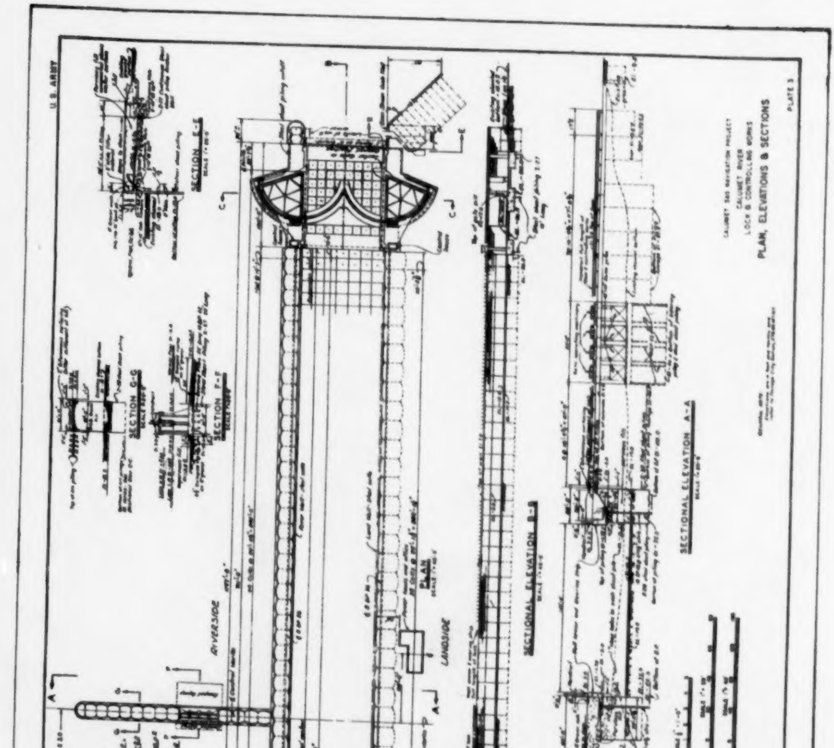
The control section of the dam is of reinforced concrete supported on steel bearing piles. Piles are required in this case because the foundation is in the soft overburden, well above the glacial till. This control section and the gate bays of the lock are the only areas that require cofferdams for construction.

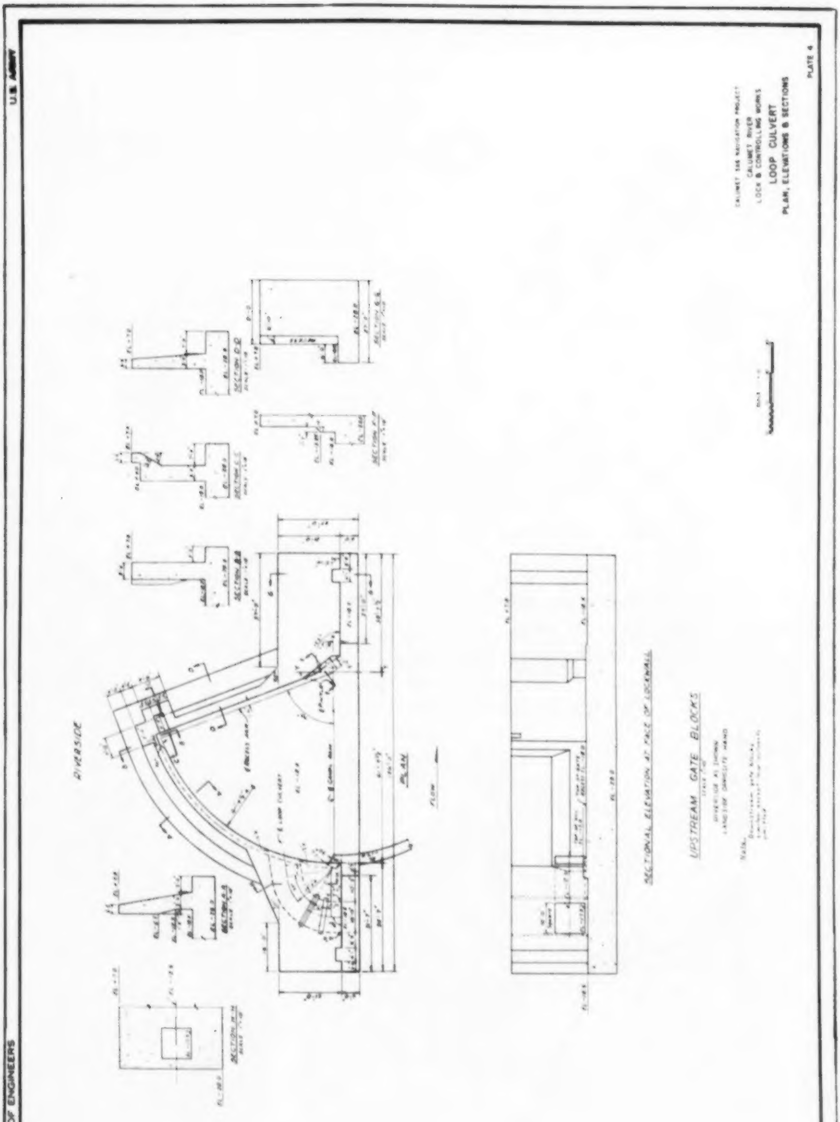
Four 10-by 10-foot sluice gates are provided to control river levels and diversion of flows from Lake Michigan. These gates are identical to the culvert gates in the lock, both in construction and operation. Recesses are

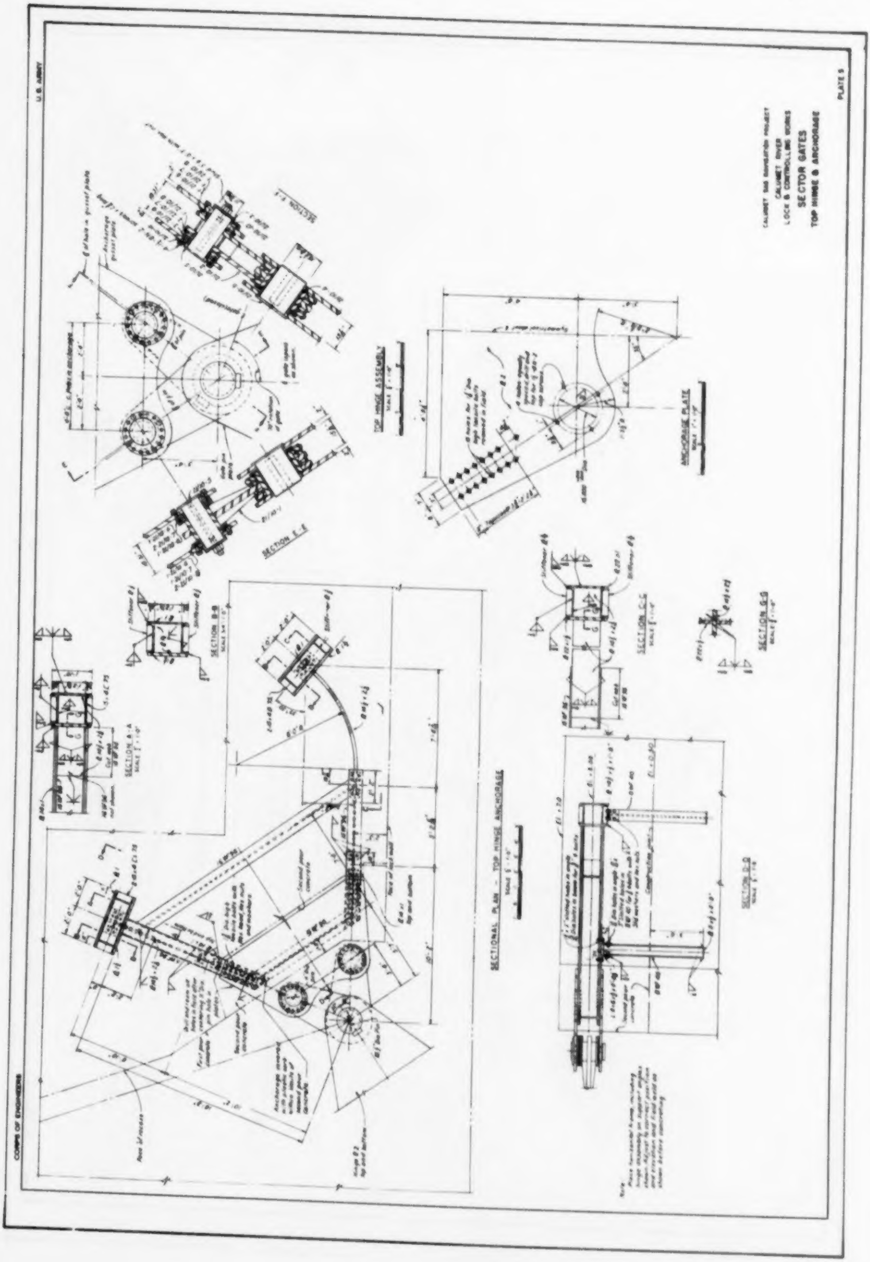
provided on both sides of these gates for bulkheads to permit unwatering of the gates. As in the case of the lock culvert, these bulkheads are to be placed by floating plant.

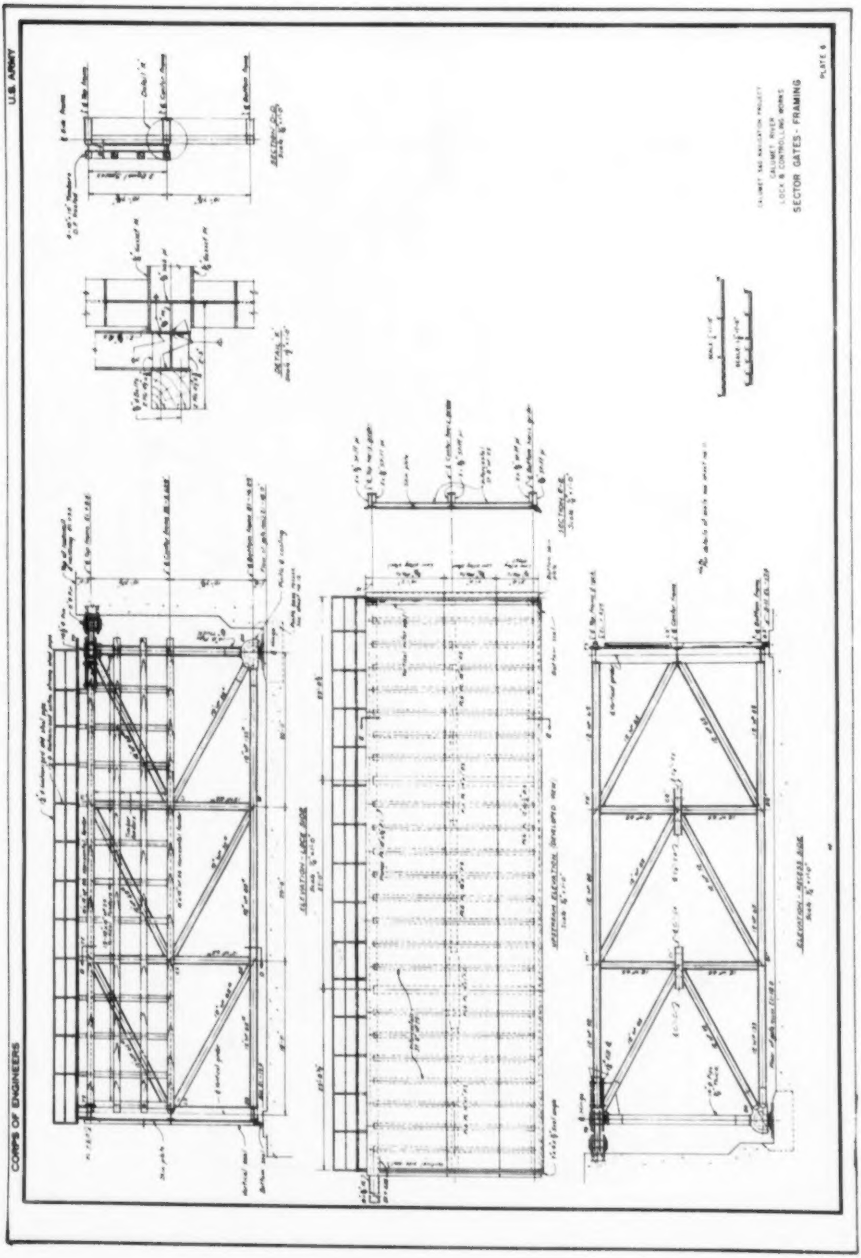

LOCATION OF LOCK AND
CONTROLLING WORKS
PROJECT LOCATION
VICINITY MAP
LOCALITY MAP
LAKE MICHIGAN
LAKE CALUMET
CHICAGO
CORPS OF ENGINEERS
PLATE 1

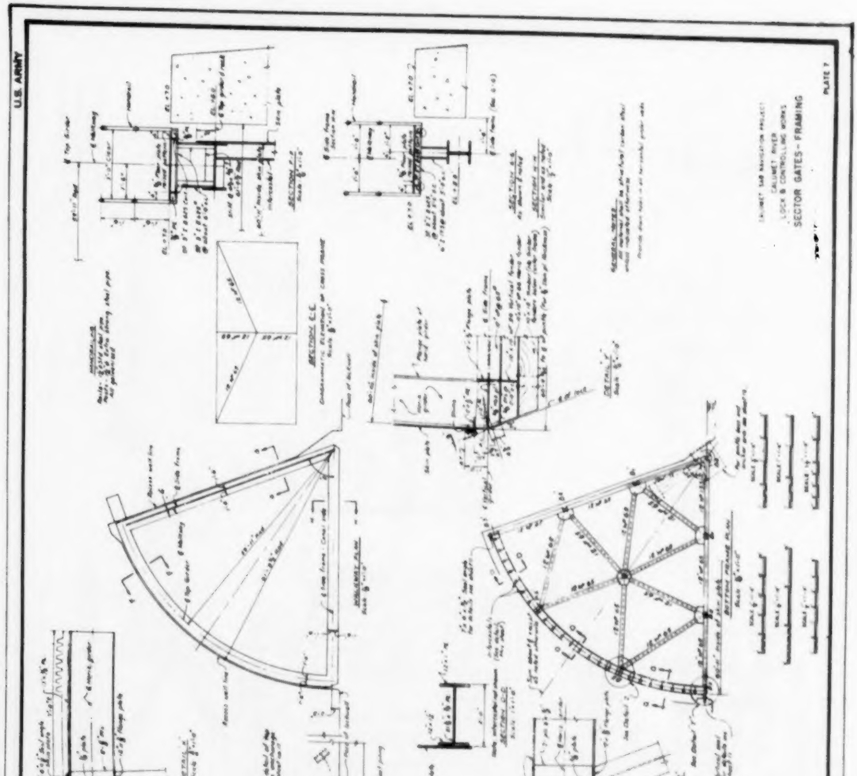

CALUMET
RIVER
LITTLE
CALUMET
RIVER
Calumet Expressway
GRAND
CALUMET RIVER
SITE MAP

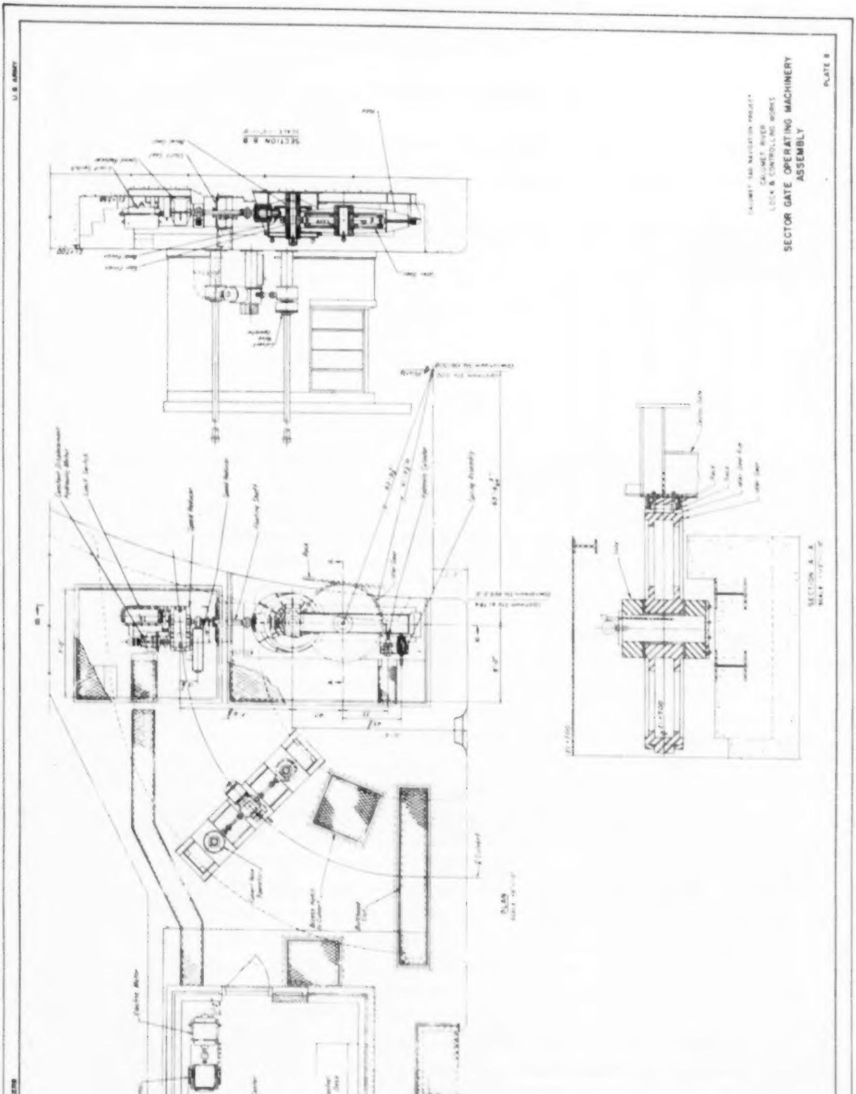

U.S. ARMY
CORPS OF ENGINEERS
RIVERSIDE
LANDSIDE
PLAN
SECTIONAL ELEVATION A-A
SECTIONAL ELEVATION B-B
SECTIONAL ELEVATION C-C
SECTION D-D
SECTION E-E
SECTION F-F
SECTION G-G
DETAIL OF WALL ARMOR AND WEARING STRIP
CALUMET SAG NAVIGATION PROJECT
CALUMET RIVER
LOCK & CONTROLLING WORKS
PLAN, ELEVATIONS & SECTIONS
PLATE 3


U.S. ARMY
CORPS OF ENGINEERS
RIVERSIDE
PLAN
FLOW
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION F-F
SECTION G-G
SECTIONAL ELEVATION AT FACE OF LOCKWALL
UPSTREAM GATE BLOCKS
CALUMET SAG NAVIGATION PROJECT
CALUMET RIVER
LOCK & CONTROLLING WORKS
LOOP CULVERT
PLAN, ELEVATIONS & SECTIONS
PLATE 4

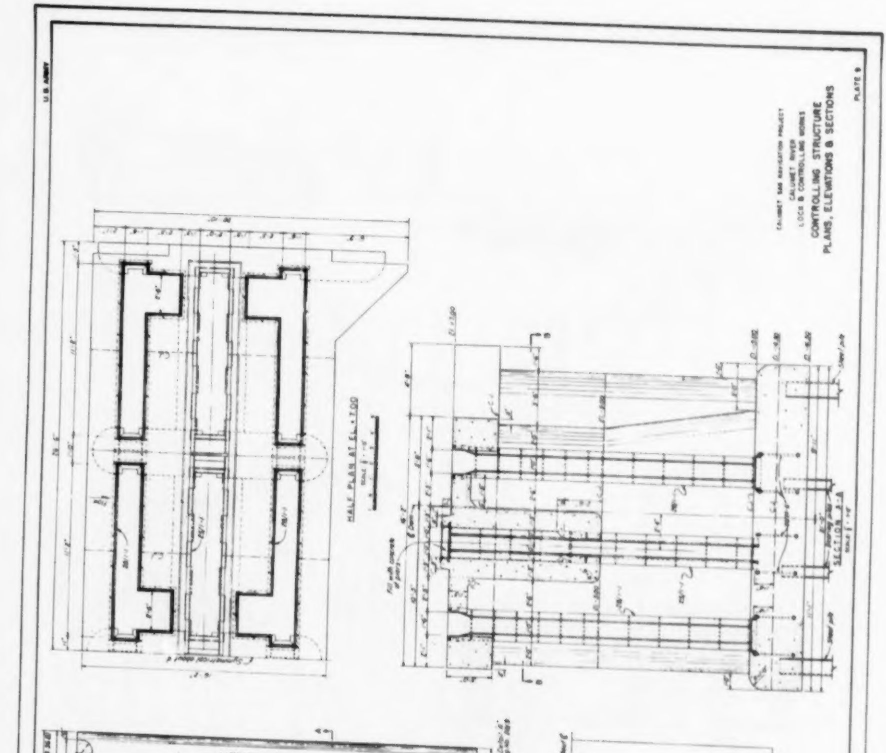

U.S. ARMY
CORPS OF ENGINEERS
PLATE 5
CALUMET SAG NAVIGATION PROJECT
CALUMET RIVER
LOCK & CONTROLLING WORKS
SECTOR GATES
TOP HINGE & ANCHORAGE
TOP HINGE ASSEMBLY
ANCHORAGE PLATE
SECTIONAL PLAN - TOP HINGE ANCHORAGE
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F
SECTION G-G


U.S. ARMY
CORPS OF ENGINEERS
CALUMET SAG NAVIGATION PROJECT
CALUMET RIVER
LOCK & CONTROLLING WORKS
SECTOR GATES - FRAMING
PLATE 6
ELEVATION - LOCK SIDE
UPSTREAM ELEVATION (DEVELOPED VIEW)
ELEVATION - RECESS SIDE
SECTION D-D
SECTION E-E
DETAIL "A"


U.S. ARMY
CORPS OF ENGINEERS
PLATE 7
CALUMET SAG NAVIGATION PROJECT
CALUMET RIVER
LOCK & CONTROLLING WORKS
SECTOR GATES - FRAMING



CALUMET RIVER
LOCK & CONTROLLING WORKS
SECTOR GATE OPERATING MACHINERY
ASSEMBLY
SECTION B-B
SECTION A-A
PLAN
U.S. ARMY
CORPS OF ENGINEERS


U.S. ARMY
CORPS OF ENGINEERS
CALUMET SAG NAVIGATION PROJECT
CALUMET RIVER
LOCK & CONTROLLING WORKS
CONTROLLING STRUCTURE
PLANS, ELEVATIONS & SECTIONS
PLATE 9
HALF PLAN AT EL. +7.00
SECTION A-A
SECTION B-B
PLAN - ANCHOR BOLT SETTING
PLAN
END ELEVATION

Paper 1643 May, 1958 WW 3

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

CALUMET - SAG NAVIGATION PROJECT[a]

John B. W. Corey, Jr.,[1] M. ASCE

(Proc. Paper 1643)

ABSTRACT

A serious obstacle to navigation on the Illinois Waterway will be removed when improvements to the Calumet - Sag Channel connecting the Chicago Sanitary and Ship Canal and the Calumet River have been completed. Barge tows four times larger than present tows will then be able to move freely through the channel to connect with deep draft navigation at Lake Calumet.

The Calumet - Sag Navigation Project is the title most frequently given the current construction program which will improve an existing connection between the Illinois Waterway and Lake Michigan and also provide an additional canal to serve industry in the vicinity of Indiana Harbor, Indiana. The enlarged waterway will accommodate barge traffic only since the project is intended to provide safe navigation for vessels drawing up to 9 feet of water. Lake vessels and ocean vessels which enter the Great Lakes by way of the St. Lawrence Seaway and the connecting channels between the Great Lakes must discharge their cargoes at deep water port facilities and take on cargo which has been brought to the deep water ports by barge, rail or truck.

Actually, the overall project includes improvement of the Chicago Sanitary and Ship Canal, the Calumet - Sag Channel and the Grand Calumet River. Construction work has commenced on the Calumet - Sag Channel only. This channel is about 15 miles south and southwest of the downtown section of Chicago. It is a part of the Illinois Waterway system which provides a channel for barge navigation between the Mississippi River and Lake Michigan and is also a part of the waterway system of the Port of Chicago.

Note: Discussion open until October 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1643 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 3, May, 1958.

a. Presented at the Chicago Convention of the American Society of Civil Engineers, February, 1958.

1. Col., Corps of Engrs., U. S. Dept. of the Army, Dist. Engr., U. S. Army Eng. Dist., Chicago, Ill.

Early settlers of the region in the vicinity of the southern end of Lake Michigan envisioned a canal system that someday would link the Great Lakes to the vast Mississippi River system. This link first became a modern reality when the Metropolitan Sanitary District of Greater Chicago constructed the Chicago Sanitary and Ship Canal. This first canal was built primarily to serve as a means of diluting and diverting sewage away from the Lake since that was the source of the city's water supply. As a later development, in order to drain the southern part of the city and surrounding area and to divert the sewage load therefrom away from the lake, the Calumet - Sag Channel was commenced in 1911 and completed in 1922.

As the city grew in size contamination spread down the Illinois River. It became apparent that some sort of treatment would have to be given the effluent from the sewers in the Chicago area. To overcome this situation, the Sanitary District began the construction of plants for the treatment of sewage in the 1920's and gradually as the sewage treatment became more effective, the contamination of the Illinois Waterway decreased.

The Illinois Waterway between the Lockport lock and dam near Joliet, Illinois and Grafton, Illinois, where it joins the Mississippi River, permits barge tows consisting of eight barges two abreast and four long propelled by a towboat to navigate and pass other tows of the same size without difficulty. The channel has a minimum width of 300 feet and provides vertical clearance of 57 feet under bridges. Above Lockport, however, the channel narrows to 160 feet in the Sanitary Canal and to 60 feet in the Cal - Sag channel, with vertical clearances under bridges as little as 14 feet. Consequently, barge tows navigating above Lockport have to be broken down into small segments which require utilization of smaller towboats providing retractable pilot houses in order to meet the clearance restrictions.

While the Sanitary and Ship Canal originally carried the major portion of waterborne traffic, the tremendous industrialization of the Calumet area led to the development of traffic on the Cal - Sag channel. In spite of the restrictions imposed, waterborne traffic grew steadily from 43,000 tons in 1935 to over a million tons in 1944.

Studies to improve the Cal - Sag channel began as early as 1924. Several reports were prepared for the Congress of the United States by the Corps of Engineers; however, they were not acted on favorably. However, a report prepared by the U. S. Army Engineer District, Chicago, in June 1945, was acted on favorably by the Congress of the United States and became the approved project for the improvements which are currently under way. The project is stated in House Document No. 677, 79th Congress, Second Session.

Description of Work to be Done

House Document No. 677, 79th Congress, Second Session, is the authority for the improvement of the Cal - Sag project and is the basis for appropriations provided by the Congress of the United States. (See map, Fig. 1, showing the project as authorized by the River and Harbor Act of 1946.) The improvement is being planned in three parts. Part I consists of the widening from its present width of 60 feet to a bottom width of 225 feet of the existing Cal - Sag channel from its junction with the Sanitary and Ship Canal to the Blue Island Lock just east of the junction of the Cal - Sag channel with the Little Calumet River and of certain straightening of the Little Calumet River

as far as Turning Basin No. 5 in the vicinity of Lake Calumet. A lock is also provided in the vicinity of 130th Street to replace the existing controlling lock at Blue Island. This lock is to be 110 feet wide by 1000 feet long with a minimum of 14 feet of water over the sills.

All of the railroad bridges across this stretch of the improvement, of which there are 18, are to be rebuilt to provide horizontal clearances of 225 feet and vertical clearances of 25 feet. There are also 13 approach structures which require rebuilding.

Of the 27 highway bridges which cross Part I of the project, only two require no alteration and six require alteration of the abutments in order that the bridges may be converted to lift bridges at a later date, if ordered. Eleven are to be rebuilt and eight are to be removed and not replaced. These bridges will provide the same clearances as the railroad bridges, i.e., 225 foot horizontal and 25 foot vertical clearance above Chicago City Datum.

Part II of the project consists of widening of the Grand Calumet River from its junction with the Little Calumet River as far east as Clark Street in Gary, Indiana and a connecting link northward from the Calumet River to the Indiana Harbor Canal at 141st Street. These streams are not navigable at the present time. This part of the project also involves the construction of a control lock and the replacement of numerous highway and railroad bridges. Whereas all bridges over the Cal - Sag channel and Little Calumet River will be fixed, the railroad bridges over Part II of the project will have to be movable bridges due to the physical difficulty and excessive cost involved in providing the required vertical clearances with fixed structures.

Part III of the project consists of the widening of the Sanitary and Ship Canal from Lockport to its junction with the Cal - Sag channel. This channel is presently 160 feet wide and will be widened to 225 feet. As in Parts I and II, the highway and railroad bridges across the channel will be altered to meet project specifications. Included as part of this project is the modification of two bridges across the Des Plaines River in Joliet, Illinois.

Part I of the project is the only part on which construction has commenced and detailed design work has been performed. The only information available on Parts II and III is general in nature which means that a considerable number of details remain to be resolved before those two elements of the project can be commenced.

Major Aspects of Part I of the Project

Major aspects of Part I of the project are:

1. Real estate acquisition.
2. Utility relocation.
3. Dredging, straightening and widening of the existing channel.
4. Relocation of railroad bridges.
5. Relocation of highway bridges.
6. Construction of the navigation lock.

Real Estate Acquisition

The Congress of the United States authorized this project on the basis that local interests would furnish certain assurances. These assurances are normally applicable to all navigation projects for which federal funds are used in the prosecution of the work. They are as follows:

1. Furnish free of cost to the United States all lands, easements, rights-of-way and spoil disposal areas necessary for the new work and subsequent maintenance, including the alteration of utilities.
2. Removal, alteration, or reconstruction of street and highway bridges.
3. A barge-rail-truck terminal facility open to all on equal terms must be provided without cost to the federal government in order to utilize the improvement.

Providing the land necessary for a project of this magnitude is a tremendous undertaking. It practically requires an organization in existence with an adequate staff with the capability of handling the funds required to obtain the necessary real estate.

Since the Metropolitan Sanitary District of Greater Chicago was the owner of the land itself, as well as certain right-of-way on either side of the canal, it was incumbent upon the Sanitary District to at least provide easements for a part of the land required for the Calumet - Sag channel widening. Actually, approximately 75% of the land required was owned entirely by the Sanitary District or by the Forest Preserve of Cook County. The Sanitary District, in addition, agreed to undertake the task of acquiring the necessary title to additional land needed for channel widening and spoil of dredged material and further, assumed responsibility for alteration or relocation of utilities as required. The problem of land acquisition in the westerly six miles of the Calumet - Sag channel was relatively simple due to the fact that the Sanitary District and the Forest Preserve of Cook County owned all of the land required for the widening operation. This enabled the project to be commenced without delays which might have arisen as a result of difficulty in negotiating for privately owned lands.

Since the federal government does not desire to acquire fee title in any additional land other than for structures, permanent easements for the channel widening areas and temporary easements for the spoil disposal areas are the only interests required to be obtained to enable construction to proceed.

The original estimates of the cost of providing land for widening the remaining ten miles of the Calumet - Sag channel and relocation of utilities were found to be entirely inadequate. Furthermore, there was no source of funds which could be used to pay for this necessary local cooperation. In order to finance the land acquisition and utility relocation, the General Assembly of the State of Illinois enacted legislation authorizing the Sanitary District to issue non-referendum bonds not to exceed $6,000,000 to obtain the necessary funds. Bonds have been issued only as required to pay for acquisition and other costs.

When negotiations were commenced to purchase real estate for the widening of the canal, beginning at Mile 6, certain owners refused to sell their land for the appraised value thereby necessitating condemnation proceedings. Since funds were available to commence construction operations, it seemed undesirable to accept the delay which would have been caused if the land could not be acquired immediately. At the request of the Sanitary District, the Corps of Engineers requested the Department of Justice to file condemnation proceedings and obtain immediate right-of-entry in order that construction could proceed without delay. The Sanitary District furnished the funds necessary for deposit with the District Court to cover the appraised values of the land in question. Similar action was taken with respect to the land required for the Calumet Lock. Every possible effort is being made to plan sufficiently far in advance in order that the federal government will not be required to file condemnation proceedings in future land acquisitions.

In the case of railroad relocations, the authorizing document requires that the necessary real estate be acquired by the United States. The contracts negotiated with the railroads provide for conveyance of the rights-of-way to the railroads upon completion of the work. Real estate for the G.M. & O., Wabash and Michigan Central relocations has been acquired by direct purchase or condemnation. That required for the Blue Island complex of railroad bridges has been appraised and negotiations are presently under way. The real estate for the remaining relocations will be acquired as construction becomes imminent. It is readily apparent that the acquisition of these new rights-of-way through such a congested metropolitan area presents many unusual and difficult real estate problems.

Utility Relocation

Utilities which pass over or under the Cal - Sag channel were granted permits for such installations by both the Sanitary District and the Corps of Engineers, U. S. Army. These utilities are to be removed without cost to the federal government as part of local assurances. These utilities consist of pipelines, power trunk lines, telephone lines and sewers. The permits indicated that the owners would be required to remove the utility at their own expense if it became necessary in the interest of navigation insofar as the Corps of Engineers' permit was concerned and that they also were to be removed at their own expense if necessary in the corporate interest of the Sanitary District insofar as the Sanitary District's permit was concerned. This clause could have required utility owners to pay for all relocation costs without reimbursement. After consideration of the hardship that this would impose, the Citizen's Advisory Committee appointed by the Board of Trustees of the Sanitary District to advise the Sanitary District on real estate problems, recommended to the Board of Trustees that they adopt a formula for the reimbursement of the cost to the utility owners for the relocation of their facilities. This formula takes into consideration the unexpired useful life of the facility and reimburses the owners for that portion of the relocation cost. In some instances, relocation has actually been performed by the general contractors having contracts with the United States for certain portions of the work. The implementation of this recommendation by the Sanitary District Trustees has simplified the problem of securing the relocation of utilities.

Dredging, Straightening and Widening of the Channel

For convenience of awarding contracts, the Calumet - Sag channel was divided into five sections. Contracts have been awarded for the westerly three sections, i.e., Sections 1, 2 and 3, which total 10 miles of the 16.2 miles to be widened. Exact dates for awarding contracts for dredging operations on Sections 4 and 5 have not been established. Design work is well advanced on Section 4 and a contract will be awarded as funds become available.

Work on the first section, awarded to Mary Construction Company, Cape Girardeau, Missouri, commenced in 1955. Widening and deepening for the first three miles of the channel which was included in the first contract has been essentially completed. (See Fig. 2) All that remains to be done is shaping of spoil banks. Dredging work in connection with Section 2, contract awarded to S. J. Groves and Sons Company, and Section 3, under contract with Mary Construction Company, is progressing ahead of schedule. Section 2, a three mile reach, is scheduled for completion in March 1959 and Section 3 in December 1959.

The existing channel was not of uniform cross-section. In some reaches, both sides of the channel were walled and in other reaches, the sides were sloped. The widened channel will have a bottom width of 225 feet with general side slopes of 2 on 1 and is being dredged to -13.0 feet Chicago City Datum. For the most part, the widening is to be accomplished entirely on the south side of the existing channel, leaving the north side in its present state—either walled or sloped.

For the first 13 miles, excavated material is being or will be placed in spoil banks immediately adjacent to the channel. The widening operation requires the removal of approximately 1,000,000 yards of material per mile.

Two classes of material were listed in the bidding schedule—common and rock. In the first 3-mile section, the material to be removed consisted of a relatively large portion of rock. The amount of rock decreased in the second section and was even less in the third section. However, in Section 4, the amount of rock will be increased as compared with Section 3, and Section 5 will contain more rock than Section 4.

Spoil banks were designed to require the least practicable area. Their shape—slope of banks and height—was determined by the stability of the natural ground upon which they were to be founded and the material itself. The slope of the banks of the channel are being carried to an average height of about 18 feet above water elevation and at that point a berm 40 to 50 feet wide is being built in order to facilitate future maintenance work. Back of the berm, the spoil bank slope varies according to type of material to be deposited.

In the first section where the foundation was solid, no special measures were required prior to placing dredged material to form the spoil banks. The actual contour of the bank in this section is not firm at this time because the contractor is crushing for sale some selected material. While there is less rock in the second section, the banks in general are relatively firm enough to not restrict spoiling operations. However, due to peaty conditions which prevail in the third section, spoiling operations had to be revised. In this area where the channel slope of the spoil bank is to be located, the peaty material has been removed and rock hauled in to form the base of the spoil bank. The height of the bank has been limited in order to offset possible upheaval of highways in the vicinity.

Where possible, material to be removed has been handled with conventional earthmovers, shovels and trucks. Material under water on or the edge of the channel is being handled with large draglines. The largest dragline carries a 15 cubic yard bucket on a 180 foot boom. This rig operates very efficiently because the boom is long enough to cast material far enough to eliminate rehandling. A second dragline with a 10 yard bucket and a 160 foot boom was the first large piece of equipment on the project. Originally, its boom was only 140 feet which required some rehandling of material. The 20 foot extension to the boom considerably improved its efficiency. These two draglines are operated 24 hours per day, 6 days per week, throughout the year, since weather conditions have not slowed their operations to any appreciable extent.

After the overburden material has been removed by the most suitable means, the rock is loosened by blasting. Blasting operations are carefully controlled to prevent accidental discharge and possible damage to private property. Underwater blasting is accomplished by first drilling holes using a drill barge. Charges are immediately lowered into holes through collars swung into place over the holes; prima cord is attached to charge before

lowering and led to the shore for later detonation. Charges are detonated using electric caps attached to the prima cord after a range of holes has been charged. Holes above water level are drilled with tractor mounted rigs on a 10 to 12 foot center. Each hole is charged with approximately 100 pounds of 40% dynamite with maximum charges detonated at one time limited to 10,000 pounds. At the end of the day shift, all holes that have been charged are exploded.

The fourth section is under design at the present time. Wherever practicable, material dredged for widening operations will be placed in spoil banks as it was in the first 3 sections. In the fifth section, the channel begins to pass through the urban section of Blue Island and spoil banks could be objectionable. In this reach, material dredged from the channel must be removed to other suitable locations.

Beyond the Blue Island Lock, the channel joins the Little Calumet River and this river together with the Calumet River then provides the channel to the end of Part I at Turning Basin No. 5 opposite Lake Calumet. No further work is necessary to improve the river since the navigable portion of the river is wide enough and deep enough at the present time. However, in the vicinity of the Acme Steel Company, the river makes a horseshoe bend which is to be widened to facilitate the passage of tows. In the past, there were several possible solutions to the elimination of this bend; however, the development of the area has eliminated their feasibility due to the increased costs. The only economically feasible solution now is to widen the present channel.

Bids received on the dredging contracts have been extremely favorable. The first contract was awarded to the Mary Construction Company, Cape Girardeau, Missouri, with a low bid of $1,786,380. This was $1,509,647 under the government estimate of $3,296,027. The second contract was awarded to the S. J. Groves and Sons Company with a low bid of $2,400,200 against the government estimate of $2,774,597. The third award was to Mary Construction on a bid of $2,727,327, which was almost identical to the government estimate of $2,726,965.

In 1930, three passing points were constructed each with a length of about 3800 feet and a width of 150 feet. At that time, the unit cost of common excavation was $0.21 per cubic yard and rock was $1.25. In the first section, the unit prices per cubic yard were $0.32 for common and $0.84 for rock; in the second section, $0.60 for common and $1.80 for rock and for the third section, $0.72 for both rock and common. In the third section, the contractor thereby avoided the necessity of classifying material. In view of the great rise in construction costs, these bids are extremely favorable in comparison with the prices quoted over 25 years ago.

Railroad Bridges

All of the railroad bridges crossing the Cal - Sag channel must be reconstructed. Costwise, this portion of the project accounts for 55% of the federal cost of Part I of the project. Relocation of the railroads in the vicinity of Blue Island is perhaps the most complex engineering problem being encountered. Due to the fact that 4 railroads converge on the Blue Island area, as well as major highways, and cross the channel which is to be widened and straightened at this point, considerable discussion was necessary to explain and illustrate the proposed changes to the affected interests. A model was

considered the best solution to the problem and one was built. The decision proved to be a wise one because many aspects of the work were explained and worked out more expeditiously than would have been possible otherwise. The federal government is paying the major cost of railroad bridge replacement in general accordance with the provisions of Section 6 of Public Law 647, 76th Congress (Truman-Hobbs Bridge Act). The excellent cooperation of the railroad companies has made possible the fine progress achieved in getting this program under way.

Highway Bridges

One of the most serious bottlenecks of the old Cal - Sag channel was the low vertical clearances and narrow horizontal clearances of the highway bridges crossing the channel. The navigation companies were able to minimize the limitation imposed by restricted vertical clearances by constructing towboats with retractable pilot houses which could be lowered to pass under the bridges. These towboats are large enough for the small tows which use the channel now but they will not be able to handle the large cargo tows of the future. For Part I of the project, it has been determined that it is possible to provide fixed bridges and still have sufficient vertical clearances to accommodate towboats capable of handling tows of 8 barges for which the channel is intended. The project provides for a vertical clearance of 25 feet for fixed bridges and further specifies that all bridges must be built with piers which will permit their conversion to lift structures at some future date if necessary. The clearance to be provided in the raised portion is 40 feet. Of the 27 bridges which cross Part I of the improvement, only two require no modification since the horizontal clearance provided is greater than 225 feet and provide 40 feet vertical clearance in the fixed position. Six of the remaining bridges provide adequate horizontal and vertical clearance but require alteration of their piers in order that they may be converted to lift structures at a later date if necessary in the event of a national emergency. Of the remaining bridges, eight are to be removed and not replaced and eleven are to be reconstructed to meet the clearances specified in the project.

Under the terms of the Truman-Hobbs Act, the federal government bears a considerable portion of the cost of relocating railroad structures when necessary in the interests of navigation. This act was subsequently modified to include highway structures. However, the modification was passed after the enactment of the Calumet - Sag legislation and did not automatically become part of the Calumet - Sag legislation. At the present time, therefore, the cost of highway bridge relocations is to be borne entirely by local owners. The local owners for the most part are Cook County and the State of Illinois. At present, it is estimated that the cost to local owners for Part I of the project will amount to $11,986,000 and the cost to the federal government will be $2,290,000. Efforts are being made to modify the Cal - Sag legislation to extend the provisions of the Truman-Hobbs Act to the highway bridges of Part I. If enacted, the legislation would reduce the cost to local interests to $1,124,000 and increase the Federal cost to $12,077,000. Little has yet been done toward the design of these bridges. As many of the highway bridges will probably be built on present alignments, real estate may not be a major problem. Also, the modification of approaches is not considered to be a major problem. For these reasons, the time required to replace the bridges will undoubtedly be shorter than for the railroad bridges and should not delay the completion of the project.

The major highway bridge involved is at Western Avenue in Blue Island. If the authorization act is modified, the bridge will then become a major responsibility of the federal government. The proposed new structure is a high level bridge over the Rock Island Railroad and will provide more than 40 feet vertical clearance over the channel.

Planning for the replacement of the Western Avenue Bridge must proceed without delay since it is an important factor in the resolution of highway traffic problems at Blue Island. Cook County highway officials have commenced discussions and studies to determine where the new structure is to be built. The structure not only serves through traffic but also links two sections of Blue Island and therefore is used for local traffic. Present plans contemplate moving the structure one block east. Since it is to be an elevated structure, some properties over which it passes will be damaged. The problem is to select the route which will cause the least damage to business and still be acceptable from the standpoint of modern highway bridge engineering. This complex problem could easily become the controlling factor in determining the completion date of the project.

Controlling Lock

Flow of water through the Cal - Sag project is presently controlled by a lock at Blue Island which is 50 feet wide and 360 feet long. A new lock is to be built in the vicinity of 130th Street which will be 110 feet wide and 1000 feet long. This lock is to be located in a new position in order to better prevent reversal of flow in the Calumet River and to control the water level in a longer length of channel. Water surface is to be maintained at a -2 feet below Chicago City Datum to provide the full 25 foot clearance under bridges.

Financing and Completion

The present estimated cost of Part I of the project is $101,000,000 of which $79,100,000 is to be provided by the federal government. The federal cost is distributed as follows: Channel Widening $20,106,000; Railroad Bridge Relocation $43,732,000; Highway Bridge Relocation $2,290,000; Lock $6,865,000; Engineering and Supervision $6,107,000. Included in the cost to local interests is the cost of real estate, relocation of utilities, owners cost of railroad and highway bridge relocations.

The first appropriation for construction was passed in 1955 and additional appropriations have been passed each subsequent year. $18,942,000 has been appropriated so far. By June 1958, approximately $14,000,000 will have been expended. Based on the amount of money expended, the project will be about 17% complete. The date scheduled for completion of Part I of the project is 1962. No dates have been set for commencing work on Parts II and III. Neither has a completion date been announced for those two parts.

Economics of Project

In the report which recommended the project, the costs of the project were compared with the benefits which it would provide. The ratio of benefits is computed each year to take into consideration changed conditions as to costs and benefits. The original benefit-cost ratio was 1.98 and the latest is 2.62.

When completed, Part I will provide an adequate through waterway between the Illinois Waterway which connects with the Ohio-Mississippi Inland Waterway system and Lake Michigan. Modern barge tows presently operating on the

inland waterways will then be able to serve Lake Calumet and the Calumet River industrial area, thereby providing the advantage of low cost transportation of bulk commodities. Extensive areas are available for development as industrial sites along the route. Already, some facilities are in operation and others are in the planning stage. These developments confirm the forecasts which were presented in justification of the improvement.

Engineering Challenge

The problems encountered in this project are typical of those encountered in modern engineering in which the trend seems to minimize technical aspects and place greater emphasis on non-technical considerations which must be resolved to satisfy various interests. The role of the engineer has been expanded and now he must be a diplomat, salesman, economist and long-range planner. These additional responsibilities must be assumed by engineers in order that technical aspects are not disregarded and, most importantly, who more logically should undertake them?

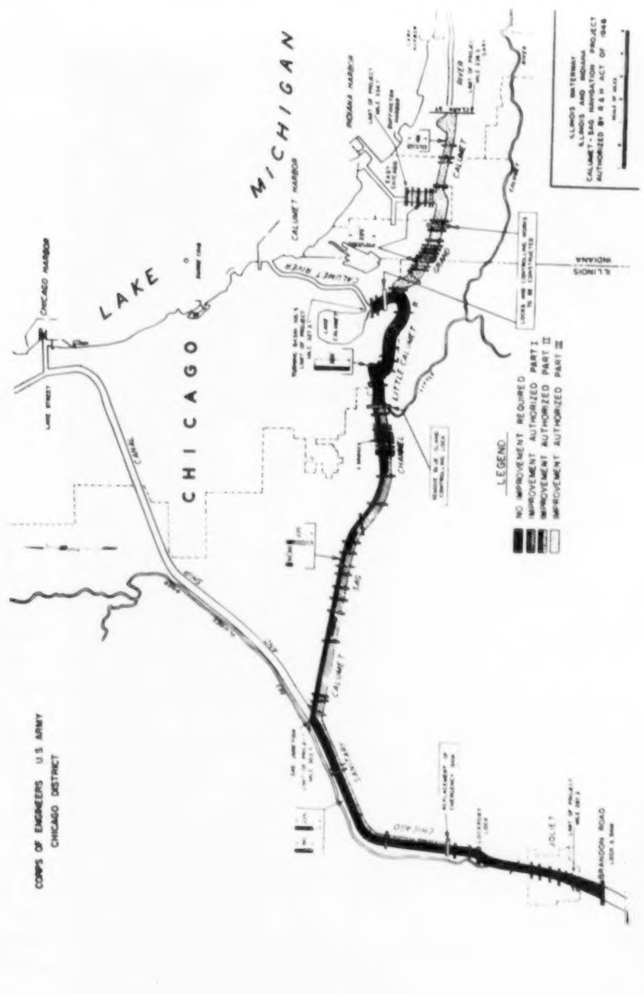


Figure 1 Project Map

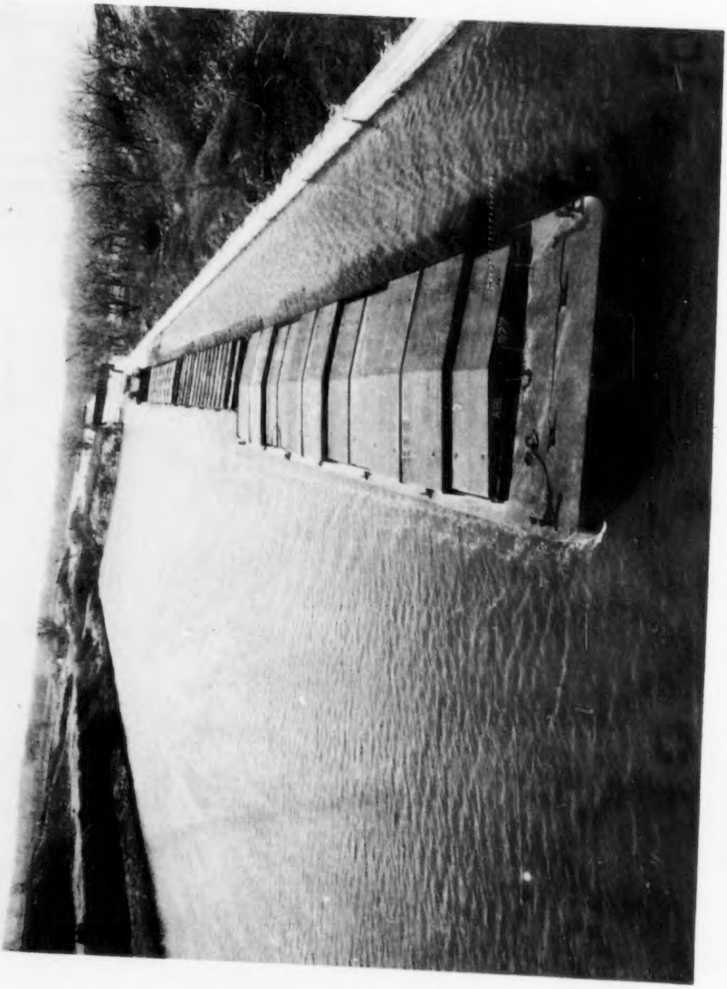


Figure 2 A Section of Widened Channel

Paper 1644 May, 1958 WW 3

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STABILITY OF COASTAL INLETS[a]

Per Bruun,[1] M. ASCE and F. Gerritsen[2]
(Proc. Paper 1644)

ABSTRACT

Existing theories of the relationship between tidal range, tidal prism, and inlet cross-section are reviewed and compared. Existing data on inlets are introduced into the discussion, as well as the effect of the quantity and type of littoral material on the inlet action.

INTRODUCTION

The birthplace of navigation was probably in some river or intracoastal waterway. Before too long man extended his interest in navigation to the oceans. The ocean activity was based in rivers and other estuaries which were the first harbors in existence.

The ancient Egyptian, Phoenician, Greek, Roman and Viking naval fleets had their bases in estuaries, fjords and lagoons—and these are the places where we have similar installations today. Now just as 3,000 years ago, the tidal estuary, river or inlet is a cultural factor of extreme importance.

A tidal inlet is the waterway connection between the sea and a bay, lagoon or river entrance. Tidal and other currents flow through the tidal inlet.

Earl I. Brown(7) discusses the origin of tidal inlets and concludes that most inlets have been cut by nature through a barrier which nature herself had built by wave action.

One can possibly distinguish between three main groups of inlets: inlets which have a geological background, inlets with a hydrological background and inlets with a littoral drift background.

Note: Discussion open until October 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1644 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. WW 3, May, 1958.

a. Presented at the New York Convention of the American Society of Civil Engineers, October, 1957.

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An example of an inlet with a geological background is the Golden Gate, the entrance to San Francisco Bay, and the Alaskan and Norwegian fjords. Such inlets have rocky gorges and do not follow normal tidal inlet physical laws.

The Thames estuary in England and the Loire estuary in France are examples of inlets with a predominantly hydrological origin. In both cases large rivers, combined with heavy tidal action, are responsible for the funnel-shaped entrances which gradually decrease in area upstream corresponding to the actual quantity of flow through the cross-section.

Fig. 1 shows some examples of such inlets and Table 1 indicates the reason why these inlets came into existence. In some cases it has been difficult to indicate one particular reason for their creation. Some of the barriers—perhaps the major part of them—are the result of wave action on the sea bottom in shallow water by which a barrier was created. Most inlets are breakthroughs, but not all inlets created in this way will "stay alive" for long because they are choked by littoral drift deposits.

In the examples in Fig. 1, three causalaties for inlet configuration are considered: Bay or lagoon geometry, direction of predominant littoral drift and direction of flood current along the seashore. Fig. 1 can be expanded very easily but this will only result in some more variations caused by a different geological background, or in inverted cases.

The examples shown are two by two symmetrical. All of them are closely related to the practical cases indicated but the inlet geometry and direction of flood current as indicated may be questioned in a few cases. The importance of inlet geometry is often not very pronounced.

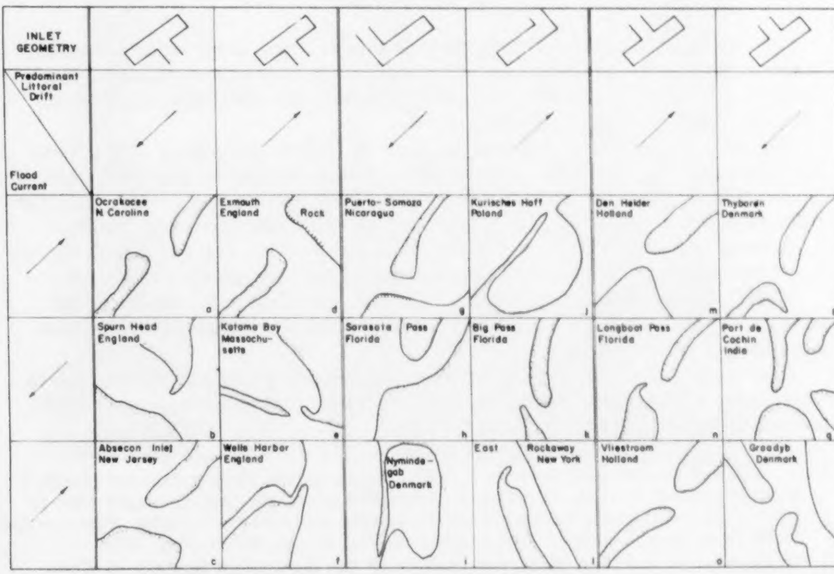


Fig. 1: Inlet Characteristics

Table 1

a.	Ocrakocee, North Carolina	Breakthrough
b.	Spurn Head, England	Formation of barrier at bay-river mouth
c.	Absecon Inlet, New Jersey	Breakthrough
d.	Exmouth, England	Formation of barrier across bay
e.	Katama Bay, Massachusetts	Formation of barrier across bay
f.	Wells Harbor, England	Formation of barrier across river mouth
g.	Puerto-Samoza, Nicaragua	Formation of barrier at river mouth
h.	Sarasota Pass, Florida	Breakthrough, possibly caused by sinking of land-mass or rise of sea level and formation of spit
i.	Nymindegab, Denmark	Formation of barrier across bay
j.	Kurisches Hall, Poland	Formation of barrier across bay
k.	Big Pass, Florida	Breakthrough, possibly caused by sinking of land-mass or rise of sea level and formation of spit
l.	East Rockaway, New York	Formation of barrier
m.	Den Helder, Holland	Breakthrough probably caused by consolidation of soil and rise of sea level
n.	Longboat Pass, Florida	Breakthrough, possibly caused by sinking of land-mass or rise of sea level
o.	Vliestroom, Holland	Probably an old river mouth now enlarged because of rise in sea level
p.	Thyborøn, Denmark	Breakthrough
q.	Port de Cochin, India	Formation of barrier in river mouth
r.	Graadyb, Denmark	Breakthrough possibly caused by sinking of land-mass and formation of spit

An entirely different method of classification of inlets is according to flow conditions in the inlet. Joseph M. Caldwell(10) divides inlets into three different classes according to flow conditions. These are:

Class 1:

a. strength of flood preceding high tide by less than one hour.
b. tidal range inside the inlet substantially equal to tidal range outside the entrance.

Class 2:

a. strength of flood preceding high tide by from two hours to three hours.
b. tidal range inside the inlet substantially equal to tidal range outside the inlet.

Class 3:

a. strength of flood preceding high tide by less than one hour.
b. tidal range inside the inlet only a small fraction of the range outside.

Class 1 entrances are backed by long estuaries and the tidal flow moves freely past the entrance. Examples of such entrances are the river estuaries: the Thames, the Loire, the Hudson and Chesapeake Bay.

Class 2 entrances include those backed by short estuaries and with tidal flow moving freely past the entrance. Examples of such entrances are Big Pass, Florida (Fig. 1, k), East Rockaway Inlet, New Jersey (Fig. 1,l), and Graadyb Inlet, Denmark (Fig. 1,r).

Class 3 entrances include those constricted sufficiently to prevent free movement of tidal flow past entrances. Examples of such entrances are Ocrakocee Inlet, North Carolina (Fig. 1,a) and Nymindegab, Denmark (Fig. 1,i).

A very special type of inlet is the river delta. An interesting theory on delta formations at river mouths is developed by Charles C. Bates.(2) It is shown that if a river delta is defined as a sedimentary deposit built by jet flow into or within a permanent body of water, these modifications allow for the existence of three basic types of deltaic deposits, depending on the density contrast between river and ocean water.

Natural Inlet Regimen

General

It is customary to talk about "nature's delicate balance" which man cannot touch without bringing about an adverse effect. The fact is that everything in nature is in a process of development and man, by interfering with this development, can influence the process in one way or another and the accompanying effects will, as a matter of course, be adverse in certain ways, but advantageous in other ways. Studying old Roman maps of the Mediterranean or the North Sea one realizes that several inlets in existence in ancient days are still "going strong" even if their location and configuration may have changed considerably. One of the most interesting examples is the Lime Inlet (Jutland), traversing the Danish mainland from the North Sea to the Kattegat. The Romans might have known about it. Ancient history tells about the Viking raids on England; of cases where naval fleets of up to 1000 ships gathered in the Lime Inlet and waited for favorable weather conditions for their passage across the North Sea to England where the Viking kingdom (King Canute) was founded. These raids took place in the 10th to 11th centuries and could hardly have been undertaken without the existence of the navigable Lime Inlet—still in existence as "Thyborøn Channel." The fact that an inlet or channel has been there for such a long time does not mean that it has been stable. On the contrary. The Lime Inlet has probably been closed for long periods and most inlets on littoral drift coasts are in a state of "dynamic equilibrium" by which littoral drift and current forces are trying to find a balance. The possibility of an inlet being "stable" depends on different factors. One has relation to the way in which the inlet was formed. As pointed out by Brown(7) inlets formed by the closing in of a bay by a spit or recurved spit will often, at least in their early stages, exhibit more favorable characteristics for navigation than those formed otherwise. They will not be affected to the same extent as the "breakthrough inlets" by inner and outer bars and shoals. Inlets usually show a tendency for migration in the direction of the predominant littoral drift if the migration is not prohibited because of rock formation. Meanwhile, if migration of such inlet stops, e.g., if the inlet is stabilized by jetties, then the inlets will often be plagued with shoaling. It is an interesting phenomenon that coastal inlets will often try to place themselves where offshore conditions are most favorable for maintenance of an inlet, as e.g., San Diego and Coos River.

Before entering into a discussion on the stability of an inlet on a physical basis, it may be worthwhile to see how inlets shoal. Any tidal inlet on a littoral drift coast is, as mentioned already, in a state of "moving equilibrium" because the conditions of flow, waves and littoral drift are always changing. Most inlets are plagued with one or two shoals, one on the sea side or one on the bay side, or both. Continued shoaling will choke the inlet if the flow does not increase in time with the shoaling effect or if the inlet is not dredged and/or protected from shoaling by jetties.

Due to the fact that the sea-shoals or offshore bars are much more exposed to wave and current action than bay-shoals, the sea-shoals are commonly stunted in their growth and margined by contours of simple curvature. Bay-shoals are apt to acquire appreciable size and the lobate form of an ordinary delta. A typical example of this is the Ocrakocee Inlet in North Carolina (Fig. 1, a).

The dimensions of the delta formation outside a tidal inlet are a function of the flow capacity of an inlet (the tidal prism). Large inlets have extensive bar formations outside. Sea-shoals as well as bar-shoals are usually formed by a number of channels the configuration and number of which is a function of the mechanics of sand transport in the inlet and flow conditions.

The mode of sand movement in an inlet is briefly explained that during flood-tide, littoral drift material is transported in the landward direction by the flood currents and deposited on shoals inside the inlet. Depending on the inlet and shoal configuration and the depth of the bay part of this material is returned in a seaward direction by the ebb currents, while other parts of the material (especially the finer grain sizes) always remain inside—being deposited at the landward end of the channel, building up the inner shoal.

The coarser material returned to sea will build up the outer bar or shoal. This shoal among other things, defers from the inner shoal in the way that the material is not as permanent here as it is on the inner bar. The outer bar or shoal will mostly establish a natural "by-passing sand plant" upon which material will pass across the inlet eventually after having been deposited momentarily in the inlet channel and pushed out by the ebb-current, to be used for building up the outer shoal or for delivery back to the normal littoral drift or both.

For elementary hydraulic reasons the tidal currents will generally be more concentrated on the discharge side of the inlet channel. This means that flood current will be more concentrated on the bay side and less concentrated on the sea side than the ebb-current—and vice versa.

One can distinguish between "flood channels" and "ebb-channels," where flood channels are predominant on the bay side while ebb-channels are predominant on the sea side. Certain channels may carry both flows equally and can be classified as "neutral." The inlet gorge if clearly defined in nature is an example of such "neutral channel." A description of the mechanism of flood and ebb channels has been given by Van Veen.[36]

Because of the situation bay-shoals will tend to develop predominantly along the flood channel(s) while the sea-shoal will develop predominantly—but less pronounced—along the ebb channel. For both cases it is valid that a continuous extension of the channel will increase the resistance against flow and when this resistance has reached a certain point new channels may break through thereby creating the half-moon shaped shoal which because of the wave action is more stunted on the sea side. This development is most predominant on the bay-shoal. As regards the sea-shoal channel its location as

pointed out by Van Veen[35] depends on the direction of propagation of the tidal wave. Other pertinent factors are the longshore current velocity, the phase difference between tidal currents in the inlet and in the ocean and the direction and magnitude of the littoral drift.

Most inlets on littoral drift coast migrate in the direction of the prevailing littoral drift. A very few inlets move in the opposite direction. The latter is the case with the now closed Indian River in Delaware and with the newly stabilized Thorsminde Inlet on the Danish North Sea coast. In both cases the predominant littoral drift is southward but the inlets tend to migrate northward. Both exceptions can be explained by special combinations of tide, flow and wave action.

The rate of migrating inlets on sandy coasts depends on the magnitude of littoral drift and the velocity of tidal and other currents. As a result of sand deposits—greater on one side than the other—the inlet channel is forced against the downdrift side and consequently causing excessive erosion (Fig. 1, a, c, b.). By such process the spit of the updrift side barrier may extend outward and in front of the downdrift side barrier or land area so as to overlap it (Fig. 1, f, i). Such a situation will usually be short lived. Most likely the inlet will be closed and a new break-through will result, in some cases because of overflooding of the barrier from storage water on the bay side and in others as a result of wave and current action. Examples of such developments are Little Egg Inlet in New Jersey and Aransas Pass in Texas. On the other hand some of the big inlets in the Baltic, such as Stettiner Haff, Frisisches Haff and Kurisches Haff have developed satisfactory and stable conditions for navigation in this way (Fig. 1, b, j).

In some cases the whole inlet migrates, whereas in other cases the gorge of the inlet keeps its place, and the tidal channels through the outer bar migrate only. Such is the case at Grays Harbor, Washington, where the migration of the channels takes place in a direction opposite to the littoral drift.

Reasons Why Inlets Shoal

It is clear that there are different reasons why inlets on littoral drift coasts shoal. In most cases the reason is of hydraulic nature, that is that the tidal prism decreases because of increasing resistance against the tidal flow—a "chain process."

Reasons in general are:

1. Prolongation of the inlet channel or channels.
2. Overwhelming supply of littoral drift material during serious storms.
3. The dividing of the main inlet channel into several channels.
4. Change in bay-area from which water flows to the inlet.
5. Other special reasons or combinations of reasons.

Most inlets shoal as a result of continuous prolongation of the inlet channel whether this prolongation takes place in the sea (Fig. 1, k, n); in the bay (Fig. 1, a, p); or parallel to the shore line (Fig. 1, f, i). Other inlets shoal very quickly as a result of violent storms during which excessive amounts of deposited sand encroach on the inlet channel, decreasing the tidal prism.

As a result of storms, new channels may develop quickly, due to an increase in the migration process of the tidal channels. With respect to navigation, this situation is unfortunate, because the old and new channels together will have about the same tidal prism as the original one. This means less

cross-section and depth for each of them if compared with the original single channel.

With respect to change in bay-area, the problem of "bay-fills" along Florida's east and west coasts should be considered. A decrease in the bay area by filling always causes a decrease in flow and tidal-prism of the connected inlets.

In the case of very long bays, as of the "Zuiderzee" in the Netherlands, the cutting off part of the bay causes an increase in the tidal prism of the inlet, a result of reflection and phase displacement of the tide.

In regard to "special reasons," it seems worthwhile mentioning the shoaling effect, often of great magnitude—which occurs where fresh water flows to the sea through an inlet. Because of the difference in specific density between debris laden salt water and fresh water, there is a tendency for the salt water to scoop itself in the bay below the fresh water which in turn will often cause severe silting. This is the case with several river inlets. Examples are the Delaware Estuary and Charleston Harbor in the United States[32] and the Ostende and Zeebrugge harbors in Belgium.[24]

Existing Results of the Stability of Rivers and Inlets

Existing results of river and inlet studies are partly of empirical; partly of theoretical nature.

Empirical River Studies

Much work has been done with respect to the stability of channels in alluvium, the results of which were to be used in irrigation design. Although the factors governing the stability of alluvial channels and tidal inlets differ in several ways, the results of the alluvial channel studies show certain similarities to the results of the study of tidal inlets.

The historical development of the alluvial channel studies is described by E. W. Lane.[25]

In 1930, Gerald Lacey[23] developed the formula:

$$P = 2.688\ Q^{0.5}$$

in which Q = Flow, or rate of discharge, and P = Wetted perimeter (not including water surface). He also developed the formula:

$$Qf^2 = 3.8\ V_o^6$$

in which f = Silt factor = $8\sqrt{D}$ when D is diameter of particles, Vo = critical velocity from the standpoint of silting. Replacing Vo by $\frac{Q}{A}$ when A equals the cross-section, have:

$$A = \sqrt[6]{\frac{3.8}{f^2}}\ Q^{5/6}$$

C. R. Pettis from data on the Miami River got the relationship:

$$A = 1.25 \cdot Q^{0.8}$$

A and Q are the same units as mentioned above. T. Blench has developed the relationships:

$$\text{(a)} \quad W = \sqrt{\frac{b}{s}} \cdot Q^{1/2}$$

$$\text{(b)} \quad D = \sqrt[3]{\frac{s}{b^2}} \cdot Q^{1/3}$$

$$\text{(c)} \quad S = \frac{b^{5/6} \cdot s^{1/2}}{2080 \cdot Q^{1/6}}$$

W = mean width in terms of depth D, so that

WD = cross-sectional area of flow, in feet.

$b = \dfrac{V^2}{D}$

$s = \dfrac{V^3}{W}$

D = depth of flow from water surface to bed in feet.

Q = discharge in cfs.

V = mean speed of flow in fps.

(a) times (b) equals $W \cdot D$ = cross-section = $Q^{5/6} \cdot \sqrt[6]{\frac{b}{s}}$

Theoretical River Studies

The best known are the works by E. W. Lane(26) and Ning Chien(13) based mostly on investigations of bed-load transport carried out by Einstein(15) and Chien. Also known is Lorenz Straub's investigations on the effect of channel contraction works.(34)

Lane's theory is based on securing a distribution of the tractive force along the sides and bottom of the channels such that the magnitude of this force at all points will be sufficiently large to prevent sediment deposits in objectionable quantities and at the same time will be small enough to prevent objectionable scour. It is possible for a given discharge to build a "stable

channel" which has the property of impeding motion over the entire periphery. Calculations on these effects are based on certain limiting values for the tractive force. A considerable amount of work has been done on the same question in Russia (Bureau of the Methodology of the Hydro-Energo Plan) and in Germany (Nuernberg Kulturamt).

Ning Chien explains how to determine channel depth and slope to conduct a specified unit discharge and sediment load.

Lorenz Straub's formula for the Effect of Channel Contraction Works upon Regimen of Movable-Bed Streams:

$$d_2 = \frac{d_1}{(1-\alpha)^{9/4}}$$

where d_1 is the water depth at the uncontracted section; d_2 the depth in the contracted section; and α is the amount of contraction. It is possible to carry the same discharge and sediment load in a smaller cross-sectional area by only deepening the channel.

Empirical Inlet Studies

In 1884 Stevenson published a table of ratios of low water sectional areas of rivers at their function with the sea, and the tidal capacity of their estuaries and discharge of fresh water. Since then the relationship between the cross-sectional area of inlets and estuaries and the flow capacity has been the subject of many investigations.

If the cross-section below mean tide A is expressed in square feet and the tidal prism Ω in acre-feet, the following results are known from previous studies:

La Conte found:

$$\frac{\Omega}{A} = 0.695$$

with the tidal prism at spring tide.

In 1931, M. P. O'Brien(5) published the results of a study of the relation between tidal prism to channel area, based on the data of a great number of inlets. O'Brien found:

$$A = 1000 \left(\frac{\Omega}{640}\right)^{0.85}$$

in which the tidal prism is taken between mean higher high water and mean lower low water, typical characteristics of the U. S. West Coast.

From a study of the Sacramento, San Joaquin and Kern Rivers, in 1931, the following results were obtained:

$\frac{\Omega}{A} = 0.93$ for unrestricted areas and

$\frac{\Omega}{A} = 1.22$ for restricted estuaries, in which Ω is the tidal prism at mean tide.

In the studies made by O'Brien(5) and others it was taken as axiomatic that inlet-geometry was primarily a result of littoral drift, constantly encroaching on each inlet to force its closure, and tidal ebb and flood flow on the other hand flushing these deposits and preventing the inlet from closing. Although nowadays it is known that this inlet morphology is also an important factor, it is realized that there are other elements involved, e.g., those related to characteristics of tides and currents.

A semi-theoretical approach to estuarine geometry is described by Herbert Chatley.(11) The following formula was derived:

$$K = \frac{W^2 h}{Q}$$

K = the rate of converging W = width

h = tidal amplitude Q = discharge

He also developed an expression for the relation between the upper width W of a reach of a given length 1, when the lower width and discharge are known and a constant depth and velocity are assumed.

G. B. Pillsbury(30) gives an equation for the "ideal form" of an inlet estuary and shows that this equation fits for the Delaware Bay.

Tidal Hydraulic Considerations

Tides are seldom of purely gravitational origin. They are often affected by meteorological conditions sometimes to a considerable degree. Tidal hydraulics deal with storm-tides as well as normal undisturbed astronomical tides.

Tidal motion can be described as a tidal wave of great length, and with a certain velocity of propagation. On the other hand, tidal currents and the differences in water levels are governed by the same laws as regular currents of permanent nature.

Formerly these two viewpoints on tidal hydraulics were considered to be basically different, but modern analysis has shown that both methods of analysis are based on the same differential equations.

Mathematically tidal motion can be described by two differential equations, the one expressing the conservation of mass (equation of continuity); and the other expressing the equilibrium of forces and momentum in the length direction of the channel (dynamic equation).

The basic equations, which govern the motion both of tidal and non-tidal currents, as mentioned in the preceding sections, are the following:

Dynamic equation:

$$\underset{(1)}{\frac{\partial v}{\partial t}} + \underset{(2)}{v\frac{\partial v}{\partial x}} = \underset{(3)}{-g\frac{\partial H}{\partial x}} - \underset{(4)}{g\frac{v|v|}{C^2 R}}$$

Equation of continuity:

$$\frac{1}{W}\frac{\partial h}{\partial t} = -\frac{\partial Q}{\partial x}$$

$$(1) \qquad\qquad (2)$$

in which v = mean velocity in the direction of the channel
x = distance along the channel
H = elevation of water level
h = water depth
g = acceleration of gravity
R = hydraulic radius
Q = discharge
W = width of channel

As expressed by Einstein and Fuchs,(16) those equations are so general and all inclusive, that knowledge of them in itself is not a great help. Only a solution of these equations which satisfies the boundary conditions of the particular channel and of the given tide cycle and hydrographs, actually gives the water surface lines and flow velocities which the engineer needs for the design of the canals and its works. To find the explicit solution of the equation for a given set of boundary conditions involves time consuming computation work, in which iterative or numerical step-methods have great advantage.

The other method of approach is to introduce simplifying assumptions into the basic equations, of which the following are the most commonly used:

1. Average elevation of water surface constant along horizontal channel.
2. Simple harmonic tide.
3. Friction term linearized.
4. Constant rectangular channel cross-section.
5. Nonlinear terms in differential equations neglected.

Einstein and Fuchs describe the United States practice in the field of tidal calculations, mentioning the following approaches to the problem.

Parson's Harmonic Theory is based on the introduction of all five approsimations mentioned above. It was the first solution of the tidal flow equations and was used for the solution of the tidal flow problems of Cape Cod Canal.

It has much similarity with the Lorentz method, used in the Netherlands for the tidal computations made for the study of the closing in of the Zuiderzee.

Because of the linear character of the equations a solution is possible in a closed form. The linear character also permits the super-imposition of solutions of different harmonic components of the tide.

In Brown's reflected wave theory(8) linear differential equal tions are used and the effect of abrupt changes in cross-section is taken into consideration.

Pillsbury's Theory(30) is based on a method of excessive approximations, in which the corrections are based on the so-called "primary currents."

In (7) Brown develops a method, in which the basic equations are simplified furthermore. This method consists of omitting the terms (1) and (2) from the dynamic equation; in the equation of continuity, term (2) is neglected for the gorge of the inlet.

The resulting equations are:

Dynamic:

$$-\frac{\partial H}{\partial x} = \frac{v|v|}{C^2R}, \quad \text{or}$$

$$V = C\sqrt{RS} \quad \text{(Chézy - formula)}$$

Continuity:

$$\frac{\partial Q}{\partial x} = 0 \quad \text{or}$$

$$d\,\Omega = Qdt = Avdt$$

The above simplifications of the two basic equations involve the elimination of the terms which vary with t, which terms determine the wave characteristics of the solution.

Further assumptions in Brown's derivations(7) are:

a) The propagation of the tidal wave in the bay is neglected, so that high water and low water occur at the same time in all locations of the bay.
b) The inlet has a uniform cross-section and depth.
c) The tidal curves in the sea and in the interior basin are sinusoidal.
d) The basin is nearly circular, so that at any given instant the tidal plane is the same in the entire basin.
e) The length of the inlet channel is well defined.
f) The inlet channel under consideration is the only connection between the basin and the sea.
g) The fresh water draining into the basin from the upland area is inconsequential, so that the volumes of inflow and outflow during a mean tidal cycle may be assumed to be equal.

Brown derives formulae for the determination of mean range of tide in the basin, mean maximum current velocities, and mean tidal prism.

In the course of its inlet studies the Philadelphia District of the Corps of Engineers succeeded in simplifying the application of Brown's theory. The value of Ω, the tidal prism, is computed from the relationship $\Omega = 17{,}040$. Avmax, when A is the cross-section of the gorge of the inlet and Vmax is the maximum average current velocity.

Garbis Keulegan(22) gives a closer approach to the problem. He omits term(1) in the dynamic equation and term (1) in the storage equation. It is an improvement over Brown's method, because, the term $V \frac{\partial V}{\partial X}$ is taken into consideration.

With respect to schematization, his assumptions are more or less the same as Brown's. By omitting the terms with t, the wave characteristics of the phenomenon are eliminated also.

In combining the two fundamental equations, Keulegan arrives at:

for flood flow: $$\frac{dh_1}{d\theta} = K\sqrt{H}\sqrt{h_2 - h_1}$$

and for ebb flow: $$\frac{dh_1}{d\theta} = -K\sqrt{H}\ \sqrt{h_1 - h_2}$$

which are the differential equations of the surface fluctuation in the basin, if h_2 and h_1 represent the water level elevations in the sea and in the bay, 2H is tidal range in the ocean (sinus-curve), θ - phase angle, and K - coefficient of filling or repletion.

In the results obtained, the coefficient of repletion K plays a decisive role. It summarizes the effects of the channels and the basin dimensions, the roughness of the channel walls, and the period and range of the tidal fluctuations on the limits of the water level changes in the basin.

It is the opinion of the authors of this paper that Keulegan's approach is quite rational for bay areas of relatively short length which can be considered as filling basins since the propagation of the tidal wave in the bay is to be neglected.

Lorentz' method(27) belongs to the harmonic methods because of the way of simplifying the basic equations:

a) the term $V\frac{\partial v}{\partial x}$ (2) is omitted in the dynamic equation

b) the friction-term (4) of the dynamic equation is linearized.

c) the bay area and cross-section introduced as constant values.

d) in the storage equation $\frac{1}{W}\frac{\partial Q}{\partial x}$ is replaced by $h\frac{\partial v}{\partial x}$

The above assumptions result in the following equations:

$$\frac{\partial v}{\partial t} = -g\frac{\partial h}{\partial x} + Kv$$

$$\frac{\partial h}{\partial t} = -h\frac{\partial v}{\partial x}$$

By means of energy-considerations the linearization coefficient k is found to equal:

$$K = \frac{8}{3\pi}\cdot\frac{g}{C^2}\cdot\frac{V_m}{h}$$

Writing $K = \frac{2\pi}{T} \, tg^2\vartheta = w \, tg^2\vartheta$ the following solution is found:

$$H = H_1 e^{iwt - (i\frac{w}{c} + \frac{w}{c} tg\vartheta)x + i\psi_1} + H_2 e^{iwt + (i\frac{w}{c} + \frac{w}{c} tg\vartheta)x + i\psi_2}$$

$$\frac{vh}{c \cos\vartheta} = H_1 e^{iwt - (i\frac{w}{c} + \frac{w}{c} tg\vartheta)x + i\psi_1 + i\vartheta} - H_2 e^{iwt + (i\frac{w}{c} + \frac{w}{c} tg\vartheta)x + i\psi_2 + i\vartheta}$$

$$c = \sqrt{gh\,(1 - tg^2\vartheta)}$$

H_1 and H_2 represent the amplitudes of the incoming and reflected tidal wave, which have to be solved from the equation.

The linear method has been worked out by J. P. Mazure(29) for cases of important fresh water discharge.

Dronkers' Method.(14) In the prediction of tidal currents in operations such as closing in of tidal gaps or estuaries, there is a need for a method by which all terms in the dynamic and storage equations are considered without any simplification. In Dronkers' method these requirements have been fulfilled. It belongs to the group "direct methods" in which the equations are subjected to a numerical integration process. Writing the basic equations in the form:

$$\left.\begin{aligned} \frac{\partial Q}{\partial x} &= -b\frac{\partial H}{\partial t} + 2bQ\frac{\partial Q}{\partial t} \\ \frac{\partial H}{\partial x} &= -m\frac{\partial Q}{\partial t} \mp wQ^2 + 2bUQ\frac{\partial H}{\partial t} \end{aligned}\right\} \quad \begin{aligned} b &= \text{width} \\ w &= \frac{1}{C^2A^2h} \\ UQ^2 &= \text{velocity head } \frac{v^2}{2g} \\ m &= \frac{1}{gA} \end{aligned}$$

the approximation $Q = Q_0(t)$ and $H = H_0(t)$ can be substituted in the right hand members and be integrated from 0 to x. This yields the first subsequent approximations:

$$Q_I = Q_0 - b\frac{\partial H_0}{\partial t}x + 2bUQ\frac{\partial Q_0}{\partial t}x$$

$$H_I = H_0 - m\frac{\partial Q_0}{\partial t}x \mp wQ_0^2 x + 2bUQ_0\frac{\partial H_0}{\partial t}x$$

The second subsequent approximations are found by substituting the latter two equations in the right hand members of the basic equations and integrating once more:

$$Q_{II} = Q_I + \frac{1}{2} bm \frac{\partial^2 Q_o}{\partial t^2} x^2 \pm bwQ_o \frac{\partial Q_o}{\partial t} x^2 + Q_s(t, x)$$

$$H_{II} = H_I + \frac{1}{2} bm \frac{\partial^2 H_o}{\partial t^2} x^2 \pm bwQ_o \frac{\partial H_o}{\partial t} x^2 \mp \frac{1}{3} b^2 w \left(\frac{\partial H_o}{\partial t}\right)^2 x^3 + H_s(t, x)$$

Q_s and H_s denote sets of terms of lower magnitude. Continuation of the process yields Q_{III} and H_{III} but as a rule the approximation Q_{II} and H_{II} are sufficient for practical computing.

In the Netherlands numerous tidal calculations have been carried out based on Dronkers' method. The method was intensively used in predicting the tidal currents in the tidal gaps formed during the flood disaster of 1953.

Some Results of Work Done by the Coastal Engineering Laboratory of the University of Florida

General

A discussion on the natural stability of an inlet is a discussion on the pertinent factors involved—or speaking hydraulically—a discussion of the relationship between tidal flow and entrance cross-sectional area, configuration of inlet and friction and other pertinent physical elements of a tidal entrance, e.g., material load (which depends on the amount of material carried to the inlet by streams or by the longshore littoral drift). Needless to say, the material through which the inlet has been cut is also of basic importance. Most inlets with a "littoral drift background" are built up wholly or partly in granular material. On the bottom of such inlets materials of a different nature as rock or clay may be found. Because of the fact that almost all inlets of that kind migrate such material is eroded in the passage of the deepest section of the gorge and replaced by granular material when the deepest spot of the inlet moves further on. The discussion and work reported on below does not necessarily assume granular material, but includes clay and other kinds of cohesive materials. On littoral drift coasts the bottom of the inlet, regardless of what kind of material the inlets are cut in, will often be covered by sand which might be swept away occasionally during extreme conditions.

The following is a list of definitions and terminology used:

Cross-section:

It is difficult to define a distinct cross-section because the littoral drift is usually attempting to decrease the cross-sections at both ends of the inlet so that conditions change all the time depending on the amount of flow. Cross-section(s) are therefore defined in the following manner:

A_{MIN} = the minimum cross-section of the channel (the gorge). A_{MIN} is referred to as A in the report.

A_{OB} = the cross-section over the outer bar. This cross-section should be measured perpendicular to the flow line.

A_{IB} = the cross-section over the inner bar. This cross-section should also be measured perpendicular to the flow lines.

Tidal characteristics:

The tidal curve under various conditions of tide, including normal tide, spring tide and storm tide. The flood-flow and the ebb-flow will, in a more detailed study, probably have to be investigated separately, because the ebb-flow will always be more concentrated over the outer bar than the flood-flow.

Bay or lagoon area Δ :

The area to which or from which water passes through the inlet. The area will be constant only with a closed bay without other inlets and without rivers.

Tidal prism Ω:

The total amount of water that flows into a harbor, a bay, lagoon or river or out during one flood of ebb period. Speaking about different tidal stages, the spring tide is generally used for determination of Ω.

Length of inlet L:

The length of the inlet channel.

Ruggedness:

The hydraulic roughness factor (Manning's "n")

Littoral drift:

The quantity of littoral drift which corresponds to the actual situation under consideration. Because of lack of knowledge in detail about this figure an estimated average amount of littoral drift will have to be used. Further investigation into this subject has been and is being carried on by Eaton, Johnson and the Beach Erosion Board.

With reference to the situation at the outer bar, the flow at floodtide as earlier stated will almost always be less concentrated over the bar than the flow at ebb-tide. The opposite will be the case with the inner bar.

For this reason, it will probably be the ebb-flow together with the littoral drift that are the main factors in determining the size and shape of the channel cross-sections over the outer bar. Conversely, it is believed to be the flood-flow, together with the material deposited during the period of flood-flow, which are often the determining factors in the size and configuration of the channels over the inner bar.

For reasons mentioned above, it is, therefore, desirable to investigate the entire length of the inlet channel, which means the gorge itself (A_{MIN}), as

well as the cross-sections over the outer and inner bars (A_{OB} and A_{IB}).

For a study based on existing data this creates the difficulty that it is often hard to define the cross-section which is in use for flow on the inner bar during ebb-tide and on the outer bar during flood-tide because the flow in both these cases will be more or less concentrated in certain channels and be unequally distributed. More detailed investigations are desirable in order to find the relationship between gorge and bar channels under different flow and littoral drift conditions. Reference is made to French's theoretical approach to similar problems.(18,19)

The first approach to the problem takes the conditions into consideration which are characteristic for the problem as it presents itself as a balance problem between "shoaling" and "cleaning." This approach is based on the Kalinske bed-load transport formula.

The Shoal or Deposit Theory

The problem is considered in the way that deposits from the littoral drift take place in the entire inlet channel during flood-tide with the biggest grain sizes on the outer part of the channel and the smallest in the inner part. Ebb-currents released from deposits in the bay's "settling basin" sweep these deposits away again picking up an increasing amount of material on their way from the bay to the sea.

The change in cross-sectional area Δ A caused by deposits must be proportional to the net amount of material removed from the cross-section of the channel Δ M.

One has: $$\Delta A = C_3 \Delta M$$

or: $$M = \frac{A - C_4}{C_3} \tag{1}$$

with: $$M = 0, \; A = C_4 = A_0$$

Per unit width of the channel the formula gives:

$$\frac{M}{W} = \frac{A - A_0}{W C_3} \tag{2}$$

when W is the width. According to the extended Kalinske formula, the transport of bed-load per unit width of channel is:

$$\frac{q_s}{d\sqrt{\frac{\tau}{\rho}}} = 10\left(\frac{\tau}{d\gamma(S_s - 1)}\right)^2$$

when: q_s is the volume of sediment transport in cu. ft. per linear foot.
S_s is the specific gravity of the average sediment particle.
d is the d 50% diameter of the sediment particle.
t is the mean value of shear stress on the channel bottom.
p is the density of the water
γ is the specific weight of the water.

This formula is based on experiments on high rates of transport.

The shear stress is an open channel is related to the slope of the channel, S, by the relationship:

$$\tau = \gamma RS$$

when R is the hydraulic radius. S is proportional to v^2 (e.g. according to Von Mieses' expression:

$$S = \frac{V^2}{8gR}\left(0.0096 + 4\sqrt{\frac{K}{R}} + \frac{0.85}{\sqrt{R}}\right)$$

For this reason q_s is proportional to v^5.

The total amount of material which will be eroded per linear foot of bottom during a half tidal cycle is:

$$\frac{M}{W} = \int_0^t q_s\,dt = \int_0^t C_1 V^5 dt$$

when T is the tidal period = 44600 seconds. Using Von Mieses' expression (which in fact is too accurate for this purpose):

$$C_1 = \frac{10}{dg^2(S_s - 1)^2}\left(\frac{1}{8}(0.0096 + 4\sqrt{\frac{K}{R}}\right)^{5/2}$$

C_1 is practically constant under the entire tidal period.

The velocity during the tidal cycle in Keulegan's first approximation varies with time as:

$$V = V_m(\sin wt)^{\frac{1}{2}}$$

where t is the time in seconds, w is $\frac{2\pi}{t}$. V_m is the maximum velocity of the current that is achieved during the tidal cycle. Integration gives:

$$\frac{M}{W} = C_1 V_m^5 \frac{T}{2\pi}\int_0^{\pi} \sin^{5/2}\theta\, d\theta = \frac{1.432\, C_1 T}{2\pi}\; V_m^5 \qquad (3)$$

Errors introduced by ignoring the existing of limited shear stress for bed-load transport and by using the approximate expression for velocity will only change the numerical constant. The extreme error occurs in the case where

the coefficient of repletion K of the inlet (Keulegan) is very large, e.g. at inlets with small tidal prisms and large minimum cross-sections and amounts to about 40%.

Keulegan has derived the following expression relating the volume of the tidal prism, Ω to V_m:

$$V_m = C_2 \frac{\pi \Omega}{T A} \tag{4}$$

C_2 is a constant depending on the coefficient of repletion of the inlet K. Keulegan gives a table of C_2 versus K. The value of C_2 varies only from .8601 to 1 when K increased from 0.1 to 100. Introducing (4) in (3) one has:

$$\frac{M}{W} = \frac{1.432\,\pi^4 C_1}{2T^4} \left(\frac{C_2 \Omega}{A}\right)^5 \tag{5}$$

Substituting in (2):

$$A = \left(\frac{1.432\pi^4 C_1 C_3 W}{2T^4 (1 - \frac{A_0}{A})}\right)^{1/6} C_2^{5/6} \Omega^{5/6} \tag{6}$$

or

$$A \backsim \Omega^{5/6}$$

Since all factors in the square brackets are 1/6 power, the effect of variations of quantities in this bracket from inlet to inlet will be small.

Equation (6) has a form very much like the empirical relationship mentioned above for rivers as well as for tidal inlets.

A similar approach to the problem based on the Meyer-Peter and Einstein bed-load formula:

$$q_s = 5.75(\tau - \tau_0)^{3/2}$$

(12) when t_0 is the critical shear stress for bed-load transport, gives the following result (when t_0 is neglected with respect to t):

$$\frac{M}{W} = \frac{1.747\, C_1' \, T}{2\pi} V_m^3 \tag{7}$$

$$A = \left(\frac{1.747\pi\, C_1 C3 W}{2T^2 \, (1 - \frac{A0}{A})}\right)^{1/4} C_2^{3/4} \Omega^{3/4} \tag{8}$$

or

$$A \sim \Omega^{0.75}$$

The evaluation of errors is somewhat more complex in this case but does not change the above similitude.

In regard to comparison with other results and formulas, reference is made to the section below dealing with this question (Page 35 - 36).

The Gorge Theory

As mentioned above the flow and deposits conditions are not well known in detail in the inlet while flow conditions are much better defined in the gorge itself.

For this reason another approach to the problem has been sought taking the stability of the gorge itself as the starting point.

The conditions of stability can be mathematically described as follows:

Consider a section dx of the gorge (Fig. 2) and let $Q_s dt$ be the total rate of sand flow passing through section x within a time interval dt, and

$(Q_s + \frac{\partial Q_s}{\partial x} dx) dt$ the amount of sand transport which passes through section x + dx. The erosion of the section during that time is equal to the difference between sand inflow and outflow.

$$\text{Erosion} = (Q_s + \frac{\partial Q_s}{\partial x} dx) dt - Q_s dt$$

$$= \frac{\partial Q_s}{\partial x} dx \; dt$$

$$\text{Erosion per linear foot of channel length} = \frac{\partial Q_s}{\partial x} dt$$

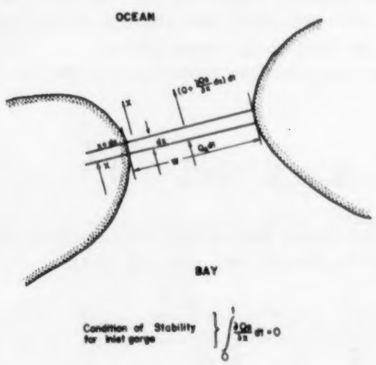


Fig. 2: Conditions of Stability

The condition of stability means that neither erosion nor depositing of material occur in the section under consideration, or, in mathematical form.

$$\int_0^t \frac{\partial Q_s}{\partial x} dt = 0 \tag{1}$$

The integral is taken over a certain length of time; in the case of subsequent equal tidal cycles, for the upper boundary t the tidal period T can be chosen.

Using Keulegan's expression for the maximum velocity in the inlet:

$$V_m = \frac{\pi \Omega}{T A} \tag{2}$$

the velocity in the inlet can be expressed by the equation

$$V = V_m \cos^p wt \quad \text{or} \quad V = \frac{C_2 \pi \Omega}{T A} \cos^p wt$$

The exponent ρ can vary for different situations.

Assuming a simple power-relationship between velocity and sand transport in the inlet:

$$Q_s = C_5 W v^n \tag{3}$$

in which the exponent may vary according to the intensity of wave action, the expression for the transport of sand in the inlet channel can be written as:

$$Q_s = C_s W \left(\frac{C_2 \pi \Omega}{T A} \right)^n \cos^{pn} wt \tag{4}$$

Now the problem is to find solutions of the equations (1) and (4). Under simplifying assumptions it can be shown that for similar circumstances equations (1) and (4) are fulfilled for a linear realtionship between and A.

$$\Omega = C_6 A \tag{5}$$

The nature of the ratio coefficient C_6 can be investigated as follows: The mean velocity in the inlet channel is related to the mean shear stress t along the wetted perimeter by the expression:

$$\tau = \rho g \frac{v^2}{C^2}$$

in which C = Chezy coefficient. The maximum shear stress therefore is equal to:

$$\tau_{max} = \rho g \frac{V_m^2}{C^2} \tag{6}$$

Introducing (2) into (6), we find:

$$\tau_{max} = \rho g \left(\frac{C_2 \pi \Omega}{C T A} \right)^2$$

or

$$\frac{\Omega}{A} = \frac{C T}{C_2 \pi} \sqrt{\frac{\tau_{max}}{\rho g}} \tag{7}$$

As could be expected, the shear stress plays a decisive part in the value of the relation $\frac{\Omega}{A}$; for this reason the shear stress value which belongs to stability conditions is called the determining shear stress or stability shear stress t_s.

For stable conditions:

$$\frac{\Omega}{A} = \frac{C T}{C_2 \pi} \sqrt{\frac{\tau_s}{\rho g}} \tag{8}$$

Further Discussion on the Configuration and Stability of the Gorge

A further discussion on the stability of the gorge must involve an evaluation of the pertinent factors determining the configuration and stability of its cross-section.

The following factors are involved:

1. The flow through and its distribution over the gorge. A study of the configuration of inlet gorges demonstrates a certain amount of similitude between the cross-section of different gorges. Meanwhile, a considerable number of inlets are provided with gorges which have not a simple univalent defined cross-section, but in the main consist of two parts, one part with comparatively shallow water and one part with deeper water (the "real gorge"). The coefficients of utilization of the different parts of the cross-section are not equal. The shallow parts have a comparatively small utilization factor for flow activity, but they count—often greatly—in the cross-section. Even if such inlets may be fairly old, it is a question of whether they can be considered as being stable because their regimen will usually demonstrate a considerable amount of instability in the form of shifting shoals and shallows.

This question is mentioned further below when the introduction of a shape factor is also discussed.

2. A stable condition of any kind always means a balance between the acting forces. In the cases of tidal inlets the stability of the bottom is obtained by a balance between the forces acting on the bottom and the forces exerted by the bottom on the flowing water. Even if the bottom is not stable in the way that it behaves like a concrete floor it must be possible to describe the stability between forces by which the flow is acting upon the bottom and the position of the bottom (regardless of whether the surface material is moving) by a force-relationship, that means hydrodynamic lift and shear stresses. The discussion below pertains to shear stresses only because it seems to be possible within certain limits to describe such stability by physical expressions including shear stresses only even this is to be understood as a combination of lift as well as tractive forces.

3. Some inlets carry silt-laden water because of fresh water and other discharge. This silt load will have an influence on the absolute size of tractive forces. At one side the specific density of the flow is increased thereby increasing the scouring ability, at another side the silt load decreases the possibility of suspension of material from the bottom. This will have an influence on the suspended load in general as well as on the material load moving as bed-load.

Next, littoral drift forces, wave and current action, will stir up material on the outer bar and this will, in particular, influence the seaward part of the inlet channel. At one side the wave action will increase the scouring ability of the flow and on the other side, the littoral drift deposits will tend to increase the determining shear stress for balance between flow and cross-section.

4. Wave action will have an influence in different ways. By the above mentioned stirring up and transport of bottom material, by increasing the transporting ability of the flow, and by the inflow of water caused by the breaking of waves over shoals by which water is mass-transported to the inlet. The latter question is probably unimportant for "stable inlets" without pronounced shoals, but it is of great importance at others. Inlets exist where, regardless of tidal range of one foot or more, water is still flowing into the inlet because of the wave breaking over shoals. The inflowing water must then return to sea through other inlets or precipitate. Some inlets on the Venezuelan coast, e.g., at Adicera behave in that way.

5. The influence of fresh water discharge is that an extra discharge of water is supplied to the normal tidal flow thereby making the quantity of flow unequally distributed between the flood and the ebb period. This question is dealt with in more detail later.

Referring to the abovementioned, it comes forth that the dimension of the cross-section of the gorge can probably be written as:

$$A = F(Q, \beta, \tau_s, f, W_a, Q_o)$$

where: $$Q = F(\Omega, T, C_2)$$

and $$\tau_s = F(\beta, f, W_a, Q_o)$$

Q = flow,

Ω = tidal prism,

T = tidal period,

C_2 = Keulegan's constant C_2 depending on the coefficient of repletion of the inlet K (22).

β = the shape factor of the gorge

t = the determining shear stress for stability of the bottom in the main channel of the gorge.

f = a silt factor.

Wa = a non-defined factor describing the influence of wave action.

Q_0 = fresh water discharge.

Referring to the considerations of the above, the (β, t_s, f, Wa, Q_0) relationship is replaced by a t-relationship in the discussion below, omitting any further discussion on the inflow of water caused by wave breaking. The relationship to be discussed is therefore simplified to the following:

$$A = F(Q, \beta, \tau_s, Q_o)$$

Of these factors Q, β and t_s are interrelated but not in any simple way.

Re Q. As mentioned earlier Keulegan found the following relationship:

$$V_m = C_2 \frac{\pi \Omega}{TA}$$

where V_m is the maximum mean velocity in a rectangular inlet channel. Keulegan explains how this simplified channel can be obtained by taking the actual cross-section into consideration. This discussion is to some extent identical with the discussion below on the influence of shape, but this discussion is rather based on a desire for classifying part of the cross-section as "useful for flow" and part of it as "useless" this because of the actual situation in practice which does sometimes not permit the introduction of any replacement channel.

Re β. Influence of the shape of the cross-section. With reference to the above discussion about the coefficient of utilization of the cross-section the following discussion is of clarifying importance.

The flow velocity is always given by the following expressions:

$$V = K \cdot R^x \cdot S^y$$

The corresponding discharge is:

$$Q = K \frac{A^{x+1}}{P^{x}} S^{y}$$

$$A = \left(\frac{Q}{K S^{y}}\right)^{\frac{1}{x+1}} \cdot P^{\frac{x}{x+1}}$$

It can be seen that the velocity for a given S is at its maximum when $R = \frac{A}{P}$ is at its maximum. The circular pipe is therefore the most advantageous for closed conduits and the half-circle for open conduits.

For a trapezoidal cross-section, it can be shown that the most advantageous (minimum) cross-section for maximum flow is one-half part of a hexagon. Of all cross-sections, the half circular is the best.

The cross-section of natural inlets differs greatly from the ideal cross-section.

In the calculations mentioned in the sections about stability, Keulegan's

$$C_2 = \frac{T Q_m}{\pi \Omega}$$

depends on the coefficient of repletion:

$$K = \frac{T\sqrt{2g}}{2\pi} \sqrt{\frac{R}{2\lambda L + R}} \cdot \sqrt{\frac{A}{H}} \cdot \frac{1}{\Delta}$$

where the relation between λ and Mannings n is given by:

$$\sqrt{\lambda} = \frac{n\sqrt{2g}}{1.486 R^{1/6}}$$

A variation with the hydraulic radius is thereby taken into consideration but the shape of the cross-section is not. As mentioned before, Keulegan to some extent converts this by introducing a "prismatic equivalence of irregular channels and multiplicity of channels."

An irregular cross-section can be split into parts. If the hydraulic radius for these parts is R_1, R_2 - - - R_n the average hydraulic radius is:

$$R = \frac{R_1 + R_2 + \cdots\cdots + R_n}{n}$$

By calculating the flow based on the single cross-sections different velocities will result because the relative roughness is not the same for each separate cross-section. In nature this unequal distribution of velocity is demonstrated not only by different velocities in the different parts of the cross-section but by eddies and transversal currents all causing a certain amount of energy loss. This again means that for any particular cross-section area a larger tidal prism is necessary because extra resistance decreases the coefficient of repletion. (See Fig. 5a where the example of Little Pass, Florida, is characteristic.)

With jetty protected "improved inlets" the cross-section is greatly improved for flow. There is only one channel with greater depth (greater hydraulic radius) and with much less "large scale" friction even if the small scale friction elements (Manning's n) are the same. Flow conditions are in other words, better "organized," for which it can be expected that a smaller tidal prism is necessary to maintain any particular cross-section.

It is interesting to note some results by Van Bendegom.(3) Based on the assumption that for a given value of water discharge and sand discharge the number of possible river cross-sections is restricted. Van Bendegom's conclusion is that for a given water discharge and sand discharge, only one stable cross-section is possible, the shape of the cross-section depending on the ratio between sand and water discharge. Fig. 3 shows the results of Van Bendegom's investigations of the Dutch "Wadden" Sea. A correlation appears to exist between the width and the depth of the different tidal channels in this area. Van Bendegom developed a theory, based on an equilibrium between the slope of the profile and the exchange of sand in a direction perpendicular to the current caused by the variation of shear stress in this direction $\frac{\partial \tau_x}{\partial y}$

According to Lane the more the actual profile differs from the "stable profile" the less stable it is. It means that material is shifted continuously in the profile. Meanwhile, for maximum flow the most ideal cross-section is the half circular cross-section, but this is not a stable cross-section such as Lane's so there is a conflict between the requirements of material-stability and maximum flow for minimum cross-section.

If a stable cross-section developed itself freely in nature maintenance operations in the gorge would be minimized. However, littoral drift deposits will always hinder this and be responsible for some irregularities on the barriers at both ends of the channel so that the ideal situation is never allowed to develop in nature.

Lane has analyzed the effect of side slopes on "limiting tractive force" by a consideration of the forces acting on a particle on the sides of the canal: the force of the water tending to move the particle down the canal in the direction of the flow and the force of gravity tending to move the particle down the sloping side of the canal.

The effect of side slopes on the critical tractive force necessary to cause motion was given as a factor β, which in the ratio of the tractive force required to start motion on the sloping sides to that force required to start motion on a level surface:

$$\beta = \cos\phi \sqrt{1 - \frac{tg^2\phi}{tg^2\theta}}$$

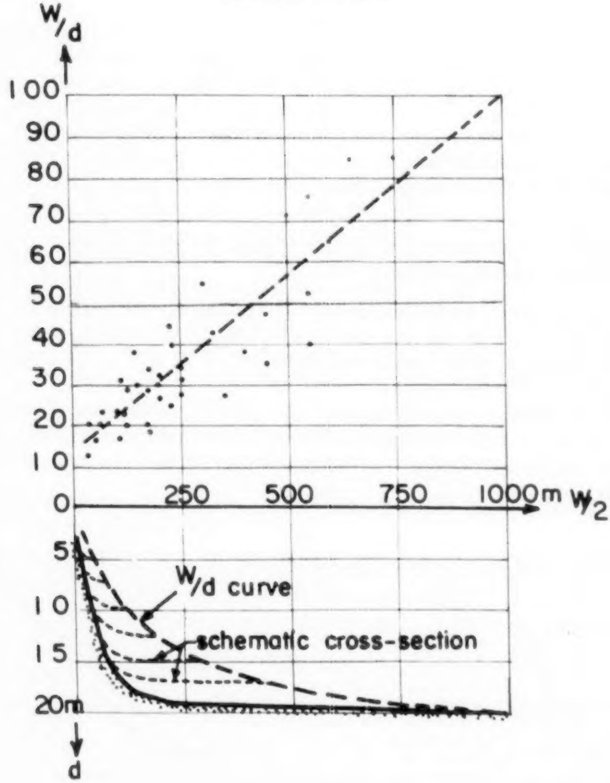


Fig. 3: Relation between width and depth of a number of tidal channels in the Dutch "Waddensea", according to Van Bendegom.

in which ϕ = angle of side slope and θ = angle of repose of the material.

Lane's approach dealing with a cross-section if simple nature not involving great irregularities in the shape of the cross-section such as flats, shoals or more than one channel, has been applied to the cross-section of some inlets of simple inlet geometry. A mean shape factor was defined as mentioned above, but for most of the inlets investigated, this factor only varied between 0.95 and 1.0.

As mentioned before, a discussion on the influence of the shape factor is a discussion on the evaluation of the cross-section including a classification of what part of it is "useful" or "practical" for flow.

In one of the preceding paragraphs it has been found:

$$\frac{\Omega}{A} = \frac{CT}{C_2 \pi} \sqrt{\frac{\tau_s}{\rho g}}$$

in which the stability shear stress t_s is a function of different inlet characteristics, as mentioned before.

The following factors will be evaluated below.

a) bottom material
b) sediment load (including contributions from the littoral drift)
c) shape of the inlet channel

Re a: The Influence of Bottom Material. Speaking about granular materials in which the major parts of inlets are cut the grain size is an important factor in the mechanism of sand transportation; it is to be understood that it is of significance in inlet geometry also.

Reference is made to Fig. 4, taken from Lane's publication, indicating limiting tractive forces, recommended for canals and observed in rivers, as a function of the mean diameter of the bed material and the amount of suspended load.

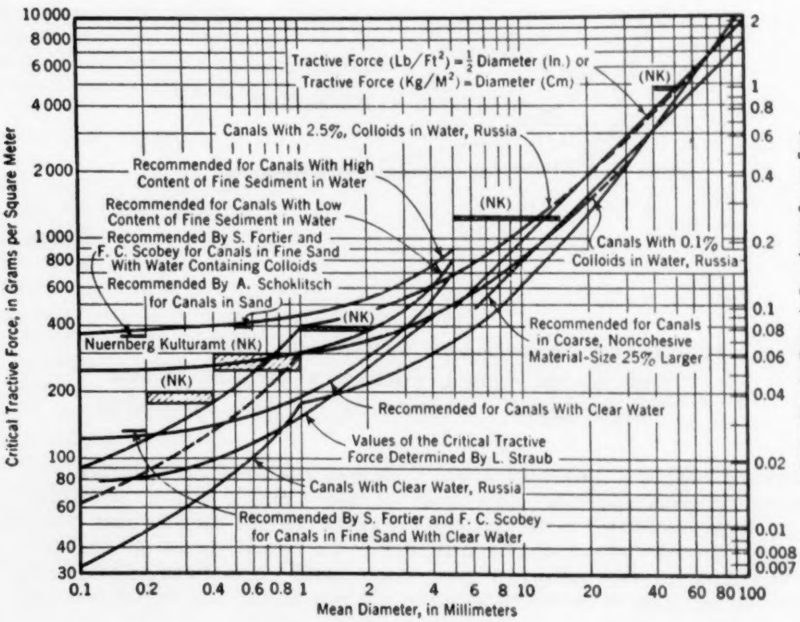


Fig. 4: Recommended limiting tractive forces for channels, according to Lane.

Many inlets on littoral drift coasts are comparable with the group "canals with high content of fine sediment in water" as represented by one line in this figure. Considering this line, the critical tractive force varies from 360 to 900 grams per square meter (about 0.08 to 0.2 pounds per square foot), when the mean diameter varies from 0.1 to 5 mm.

The sand from littoral drift coasts often has a mean diameter of from 0.1 to 0.5 mm. In this way the critical tractive force varies only to a small extent and within this range and it is probable that the variations in grain size have only a limited influence on the stability shear stress.

A comparison between the results mentioned in Fig. 4 and those by the Coastal Engineering Laboratory of the University of Florida is mentioned in the section below about results of studies of existing data.

Re b: The Influence of Sediment Load in the Flow. Sediment load of the inlet flow has its origin from three basically different sources:

1. Suspended load in the tidal flow, caused by the stirring up of material from the bed under turbulent flow conditions. (This suspended load is related to the amount of bed load, as indicated by Einstein.)[15]
2. Silt load from rivers carrying material from upland sources into the estuary or inlet.
3. Suspended material stirred up by wave action and being transported as littoral drift into the inlet channels.

In different studies dealing with stable channel profiles, the influence of sediment load is recognized as an important factor of stability.

Experiments initiated by Lacey[23] and Inglis[21] gave the following set of formulas for the relation between cross-section A discharge Q and "silt load":

$$A = 1.26 \frac{Q^{5/6}}{f^{1/3}}$$

$$V = 0.7937\, Q^{1/6} f^{1/3}$$

$$S = 0.000547 \frac{f^{5/3}}{Q^{1/6}}$$

$$SV = 0.000434\, f^{2}$$

The silt factor f in river technology must in the tidal hydraulics technology be replaced by a "sediment load" factor which will depend mainly on the littoral drift conditions including the grain size and amount of material carried to the inlet by littoral forces as suspended or bed-load.

Because of the fact that $V \backsim \sqrt{\tau}$ and $S \backsim \tau$ one has: $f^2 \backsim \tau^{3/2}$ or $f \backsim \tau^{3/4}$ This in turn means that

Table 2

Median size of material, in millimeters	DESCRIPTION OF WATER		
	Clear water	Light load of fine sediment	Heavy load of fine sediment
0.1	0.025	0.050	0.075
0.2	0.026	0.052	0.078
0.5	0.030	0.055	0.083
1.0	0.040	0.060	0.090
2.0	0.060	0.080	0.110
5.0	0.140	0.165	0.185

$$A \sim \frac{\Omega^{5/6}}{\tau^{1/4}}$$

The above mentioned shear stress formula has: $A \sim \frac{\Omega}{\tau^{1/2}}$
which shows the same kind of relation, however, with a different power of t.

Table 2 by Lane[26] indicates the recommended limiting values of the tractive forces in pounds per square foot for canals in fine non-cohesive materials such as sand.

It gives an impression of the influence of sediment load on the size of tractive forces with different median size of material indicated in millimeters.

In Table 3 the square, third and fourth root of these figures are given in order to see the influence per centage-wise.

Table 3

Median size of material, in millimeters	DESCRIPTION OF WATER								
	Clear water			Light load of fine sediment			Heavy load of fine sediment		
	$\sqrt[2]{\tau}$	$\sqrt[3]{\tau}$	$\sqrt[4]{\tau}$	$\sqrt[2]{\tau}$	$\sqrt[3]{\tau}$	$\sqrt[4]{\tau}$	$\sqrt[2]{\tau}$	$\sqrt[3]{\tau}$	$\sqrt[4]{\tau}$
0.1	0.16	0.29	0.40	0.22	0.37	0.47	0.27	0.42	0.52
0.2	0.16	0.30	0.40	0.23	0.37	0.48	0.28	0.43	0.53
0.5	0.17	0.31	0.41	0.23	0.38	0.48	0.29	0.44	0.54
1.0	0.20	0.34	0.45	0.24	0.39	0.49	0.30	0.45	0.55
2.0	0.24	0.39	0.49	0.28	0.43	0.53	0.33	0.48	0.57
5.0	0.38	0.52	0.62	0.41	0.55	0.64	0.43	0.57	0.65

Considering as an example a median grain size of 0.2 mm. the influence on the limiting shear stress (not to be confused with the stability shear stress mentioned earlier) when the "silt-load" characteristics of the water varies from "clear" to "Heavy" is a 75% rise when $2/\tau$ is assumed, a 43% rise when $3/\tau$ is assumed, and a 32% rise when $4/\tau$ is assumed. Using the abovementioned formulas a corresponding variation in cross-section should result but because of the difference between flow conditions in a unidirectional flow and a tidal flow such variation can probably not be expected. Yet it may be possible to trace such influence on the size of the cross-section of inlets when inlets on coasts with a heavy suspended load are compared with inlets with a less amount of suspended load and in particular, when a heavy silt load from fresh water flow is involved also. The result of wave action is in this way indirectly incorporated in the study without considering actual wave data. Wave action will result in more material load in the outer bar but even if the gorge itself is protected the flood-flow may carry a surplus load in the gorge and less scouring will result. Reference is made to the section below dealing with results of a study of existing data on tidal entrances.

An approach based entirely on the supply of littoral drift material to the inlet is the following:

If the average amount of littoral drift per unit of time which acts upon the tidal channel outside the gorge is **M**, a percentage (p) of it will be moved through the gorge. This part is added to the transport of sand Q_S, which is passing the gorge under conditions without littoral drift, as discussed before. This quantity, **pM**, does not influence the validity of the stability equations, but it does change the value of the ratio factor between Ω and A. The littoral drift causes the amount of sand transport per unit of time through the gorge to increase in the ratio:

$$\frac{Q_s + pM}{Q_s} = 1 + \frac{pM}{Q_s}$$

The power-relationship $Q_s = C_s W v^n$ can be written as

$$Q_s = C_s' W(\tau_s)^{n/2} \text{ and also } Q_{sm} = C_s' W(\tau_{sm})^{n/2}$$

The index m denotes the littoral drift conditions. Further:

$$\frac{Q_{sm}}{Q_s} = \left(\frac{\tau_{sm}}{\tau_s}\right)^{n/2} = 1 + \frac{pM}{Q_s}$$

and

$$\sqrt{\frac{\tau_{sm}}{\tau_s}} = \sqrt[n]{1 + \frac{pM}{Q_s}}$$

or

$$\sqrt{\tau_{sm}} = \sqrt[n]{1 + \frac{pM}{Q_s}}\ \sqrt{\tau_s}$$

In the equation above, t_{sm}, t_s, and Q_s are functions of time, whereas pM is to be considered a constant value. The value of the stability shear stress increases under influence of the littoral drift in the ratio:

$$\sqrt[n]{\left(1+\frac{pM}{Q_s}\right)^2}$$

In that case the relationship between tidal prism and cross-section will be transformed into:

$$\frac{\Omega}{A} = \frac{CT}{C_2\pi}\sqrt{\frac{\tau_s'}{\rho g}} \cdot K_1\sqrt[n]{1+\frac{pM}{Q_s}}$$

If t_s represents the stability shear stress in the case there is no littoral drift. If we call:

$$r = \frac{CT}{C_2\pi}\sqrt{\frac{\tau_s'}{\rho g}}$$

then the relationship between Ω and A can be written in the form:

$$\frac{\Omega}{A} : r = K_1\sqrt[n]{1+\frac{pM}{Q_s}}$$

Theoretically the function $K_1\sqrt[n]{1+\frac{pM}{Q_s}}$ can be computed provided the relationship between Q_s and $\sqrt{RS}$ is known in tidal flow.

The influence of suspended material on the tractive forces is mentioned briefly in the preceding paragraph.

In regard to the influence of grain size on the amount of suspended material, two factors are important on sea coasts:

a) the absolute size of the grains, and
b) the wave action.

The smaller the grains are (within certain limits), the more material will be suspended; and the heavier the wave action is, the more material will be suspended. Now the actual situation is that grain size and wave action not necessarily go together. Large waves will only select the biggest grains available at the particular locality.

On the United States East Coast, the South Shore of Long Island and the coast between Sandy Hook and Barnegat Inlet, New Jersey, has heavy wave action and a high average grain size (0.4 - 0.5 mm.) while Daytona Beach in Florida has moderate wave action and small average grain size (0.2 mm.). The Gulf Coast in general has light to moderate wave action and an average grain size less than 0.2 mm. while the Pacific coasts have moderate to heavy wave action with average grain size 0.2 - 0.3 mm.

One could therefore, expect comparatively bigger inlet cross-sections on the Pacific—and perhaps also the Gulf Coast—than on said part of the Atlantic coast. Investigations carried out by the Coastal Engineering Laboratory of the University of Florida have not yet given any indication in this respect, but on the other hand, revealed the fact that inlets on coasts with a comparatively greater littoral drift (sediment load) have a greater tidal prism when compared with the corresponding inlet gorge cross-section. This is mentioned more in detail in the paragraph dealing with results of studies based on actual data.

Re c: The Influence of the Configuration of the Inlet Channel. The influence of the configuration of the inlet channel including the shape of the cross-section on t is unquestionable. Bends and irregular cross-sections will unequally distribute the shear stresses concentrating them in some places and de-concentrating them in others. Meanwhile, such influence can partly be taken care of in the calculations of flow conditions. In the results of studies of such channels, exemptions and irregularities are always to be expected.

The Influence of River Discharge

If a river discharges through a tidal inlet, the relation between Ω and A will be changed to a certain extent. If freshwater discharge is involved, density differences create a more complex current distribution, such as described by Stommel,(33) and Schultz and Simmons.(32)

Despite the differences in flow characteristics between estuaries with and without fresh discharges, the fundamental stability equation for the gorge is still valid:

$$\int_0^t \frac{\partial Q_s}{\partial x}\, dt = 0$$

A consequence of the freshwater discharge is that flood and ebb flow have a different current distribution in the vertical direction.

Probably the relation $Q_s = C_5 W V^n$ will remain valid, but the coefficient C_5 has a different value for ebb and flood. If this difference is neglected, an approximation for the relation (Ω, A) is to be found as follows:

If Q_0 is the average river discharge, the rate of flow as a function of time is to be expressed by the formula:

$$Q = Q_0 + \frac{C_2 \pi \Omega}{T} \cos^p wt$$

and the mean velocity in the cross-section:

$$V = \frac{Q_0}{A} + \frac{C_2 \pi \Omega}{TA} \cos^p wt$$

With the assumptions above and in continuation of the shear-stress

stability approach mentioned above, the following expression for the ratio between and A can be obtained:

$$\frac{\Omega}{A} = \frac{CT}{C_2\pi}\sqrt{\frac{\tau_s}{\rho g}} - \frac{Q_0 T}{2A\xi C_2}$$

The coefficient ξ depends on the shape of the velocity curve, that means the exponent "p" of the velocity equation. It appears that the ratio between the value of A increases if freshwater discharge is to be taken into consideration. In most cases the influence of Q_0 upon the $\frac{\Omega}{A}$ ratio is very small.

Comparison Between the Different Theories: A Practical Method of Approach for Design

In the preceding paragraphs the following semi-theoretical formulae for the relation between Ω and A have been obtained.

a) Based on Kalinske's bed-load transport formula:

$$A = \left[\frac{1.432\ \pi^4 C_1 C_3 W}{2T^4\left(1-\frac{C_4}{A}\right)}\right]^{1/6} C_2^{5/6}\ \Omega^{5/6}$$

b) Based on Meyer-Peter's and Einstein's bed-load transport formula:

$$A = \left[\frac{1.747\ \pi^2 C_1 C_3 W}{2T^4\left(1-\frac{C_4}{A}\right)}\right]^{1/4} C_2^{3/4}\ \Omega^{3/4}$$

c) Based on stability requirements:

$$\frac{\Omega}{A} = \frac{CT}{C_2\pi}\sqrt{\frac{\tau_s}{\rho g}}$$

The difference in results originates in the difference in assumptions. For the expressions "a" and "b" conditions near the outer bar or where deposits mainly take place have primarily been taken into consideration resulting in a power-relationship between Ω and A.

The assumptions for the formulae "a" and "b" were a uni-directional cleaning flow, just like rivers, and the laws of sand transportation, as valid for rivers, have been applied. The similarity between those formulae and different empirical river and inlet formulae is notable although the ways these formulae have been derived are different. The question of whether this is a matter of coincidence or demonstrates that the similarity between river and

actual tidal flow is greater than expected must remain unanswered at the present moment.

Compare: Lacey's River Formulae: $A \backsim Q^{5/6}$ $(Q \backsim \Omega)$

Pettis' results from Miami River:(29) $A \backsim Q^{0.8}$

Blench's formula:(4) $A \backsim Q^{5/6}$

O'Brien's inlet formula(5) $A = 1000\left(\frac{\Omega}{640}\right)^{5/6}$

Gorge formula "c" is based on stability considerations and as mentioned in the following paragraph, it gives good agreement for the number of inlets to which it has been applied.

Referring to the results of studies by the Coastal Engineering Laboratory of the University of Florida, it is interesting to compare the gorge formula with linear relationships in literature of earlier date. If mean values for the different factors are introduced, the gorge formula gives:

$\frac{\Omega}{A} = 1.26$ - heavy sediment load

$\frac{\Omega}{A} = 1.17$ - average condition

$\frac{\Omega}{A} = 1.06$ - light sediment load

for conditions at springtide, Ω expressed in acre feet and A expressed in square feet.

LaConte found $\frac{\Omega}{A} = 0.695$ (Ω at springtide) and from a study of the Sacramento-San Joaquin and Kern Rivers (1931): $\frac{\Omega}{A} = 0.93$ for unrestricted areas (Ω at meantide) and $\frac{\Omega}{A} = 1.22$ for restricted areas (Ω at mean tide).

LaConte's results apparently were related to a situation for which a lower value of the stability shear stress was valid. (e.g. finer grain size).

Results of Study of Existing Data

After careful investigation of the material available a number of inlets were selected for further analysis. Basis of selection was the availability of different data to determine the cross-section of the inlet, the tidal prism and the velocities in the gorge with adequate accuracy. Besides a number of inlets in the United States, two inlets from Denmark and six inlets from Holland were incorporated in the study.

Most inlets which have been chosen present comparatively simple inlet geometry; inlets with one simply defined gorge with comparatively regular shape with not more than one channel, no shoals and with a simple configuration, mainly straightlined. The reason for this is the desire of eliminating shape and configuration factors involved in the problem by obtaining well-defined flow conditions and thereby a simple distribution of shear stresses over the cross-section. At the same time typical tidal inlets without any appreciable fresh water (and silt) flow have been sought. The results therefore are valid for inlets with such simpler natural conditions—and not for just any inlet.

The reason for this approach is partly the scientific desire of "splitting up in parts" and partly comparison with Lane's theory for stable channels which is based on shear stress analysis. In "Design for Stable Channels" Lane used the "limiting tractive force," but canals with movable bed are considered as well. Replacing Lane's "limiting" with the "determining" tractive force is here used as the approach to the problem.

Use of the "deposit theory" would involve difficulties in determining the inlet characteristics where these deposits take place—simply because this is not very well known and is changing all the time. The study of a number of inlets, of simpler geometrical nature on littoral drift coasts in the United States as well as in Holland and Denmark has shown, that a maximum average springtide velocity of 1.1 m/sec. and a Chezy coefficient value of 50 $m^{1/2}$ sec $^{-1}$ often go together, resulting in a stability shear stress factor:

$$\frac{V}{C} = \sqrt{\frac{\tau_s}{\rho g}} = 2.2 \times 10^{-2}\ m^{1/2}$$

Under normal tide conditions this is about 10% less, and:

$$\sqrt{\frac{\tau_s}{\rho g}} = 2.0 \times 10^{-2}\ m^{1/2}$$

which gives a computed value of t_s of 411 g/m^2 which value is almost in agreement with the line for high content of fine sediment.

In Fig. 5a and 5b the calculated values of Ω and A are plotted in a diagram. Fig. 5a represents only small inlets, whereas Fig. 5b covers inlets up to a cross-section of 900,000 ft. sq.

In order to compare the plotted values with some semi-theoretical relationships, two lines have been drawn in each of the diagrams:

O'Brien's exponential curve: $A = 1000\left(\frac{\Omega}{640}\right)^{0.85}$ and the linear relationship between Ω and A, as based on gorge stability consideration:

$$\frac{\Omega}{A} = \frac{CT}{C_2 \pi}\sqrt{\frac{\tau_s}{\rho g}}$$

if in the latter formula the following values have been introduced:

C = 90 ft. 1/2 sec $^{-1}$ = 50 m 1/2 sec $^{-1}$

V = 3.6 ft/sec = 1.1 m/sec

C_2 = 1.0

t = 44600 sec.

giving the linear relationships:

$\frac{\Omega}{A} = 1.56 \times 10^4$ m

or $\frac{\Omega}{A} = 1.17\ \frac{\text{acre}}{\text{ft.}}$

The following conclusions can be arrived at:

1. The linear relationship is fairly well in agreement with observed values for both small and large inlets, although some remarkable deviations exist.

2. The exponential formula (O'Brien's equation) gives sufficient agreement for smaller inlets (cross-section below 50,000 sq. ft.) but deviates from the observed data for the bigger inlets. The rocklined gorge of Golden Gate * has less than half the area, relative to volume of tidal prism, of inlet gorges with sand bed and banks. As expressed by Eaton it seems less reasonable to let this condition at this inlet carry the same weight as conditions at inlets developed freely in granular material. Yet it is still possible that further studies of the "very big inlets" will give the result that such inlets need comparatively smaller cross-sections. Density currents and special boundary wave phenomena between water layers may play a role.

For some smaller inlets the plottings show considerable deviations. Among these are Calcasieu Pass (Louisiana), Port Aransas (Texas), and East Pass (Florida), the tidal prisms of which are considerably higher than indicated by the average line, and San Diego and New Port, which have smaller values for Ω.

The first ones with too high values of Ω are all located on the Gulf Coast, having a pronounced diurnal tide, for which the tidal period is 86400 sec. instead of 44600 sec. Considering the linear relationships these inlets should be compared with the lines:

$$\frac{\Omega}{A} = 3.02 \times 10^{-4}\text{m} \qquad \text{or} \qquad \frac{\Omega}{A} = 2.27\ \frac{\text{acre}}{\text{ft.}}$$

and then the agreement is much more satisfactory.

San Diego and New Port belong to the group of non-scouring channels, in which a larger cross-section is created artificially by dredging. In these inlets no natural stable conditions exist and it is therefore not surprising that their plottings deviate in the diagram.

Even though we have explained the greater deviations in the diagrams of Fig. 5a and 5b, the remaining differences between the average line and the individual plottings need further analysis.

In the linear relationship between Ω and A, different values of C, C_2, T, and t_s have to be taken into consideration. The values of C, C_2 can be computed from inlet geometry, and the value of T is known also. The remaining unknown factor is t_s.

When introducing for t_s the average value

$$\sqrt{\frac{\tau_s^T}{\rho g}} = 2.2 \times 10^{-2}\ m^{1/2}$$

* as used by O'Brien

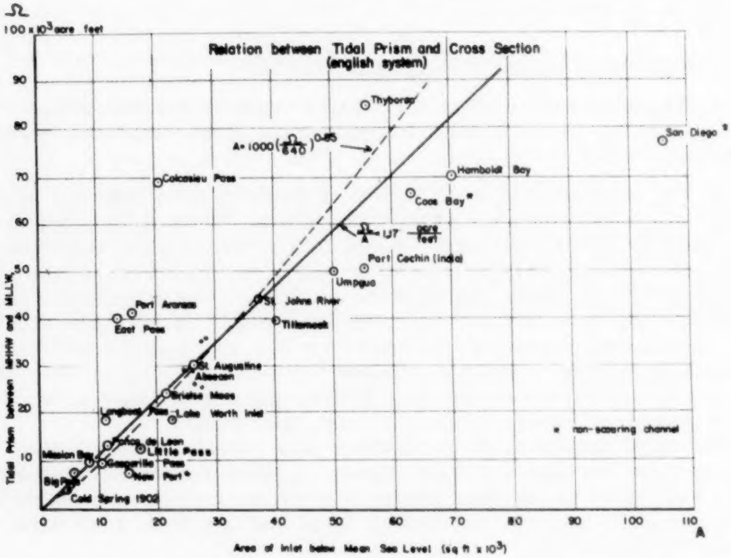


Fig. 5a: Statistical relationship between tidal prism and cross section for small inlets.

the variations in t_s can be found by comparing the plotted data and the computed line with $t_s = t_s'$. The results of this analysis are shown in Fig. 6, in which the ratio $\dfrac{\Omega}{\dfrac{C\,T}{C_2 \pi}\sqrt{\dfrac{\tau_s'}{\rho g}}}$ is plotted against the cross-section A. A dimensionless diagram has been obtained by dividing both variables by Δ, the area of the bay.

Because $A \sim \Omega$, the ratio $\frac{A}{\Delta}$ represents the reverse of the tidal range in restricted bay areas and a grouping of data has been obtained according to tidal characteristics in the bay.

From this diagram it can be seen that inlets with a comparatively higher value of Ω than indicated by the average line generally have a higher sediment load or littoral drift and for the points with lower values of Ω, the sediment load or littoral drift generally is lower.

In considering the influence of the littoral drift, it has to be realized, that not the resulting littoral drift should be taken as a value of comparison, but the total amount of littoral drift, which encroaches from both sides upon the inlet. Furthermore, the littoral drift should be introduced as a ratio factor related to the amount of sand transport Q_S, or, as a further approximation, to the maximum flow through the inlet.

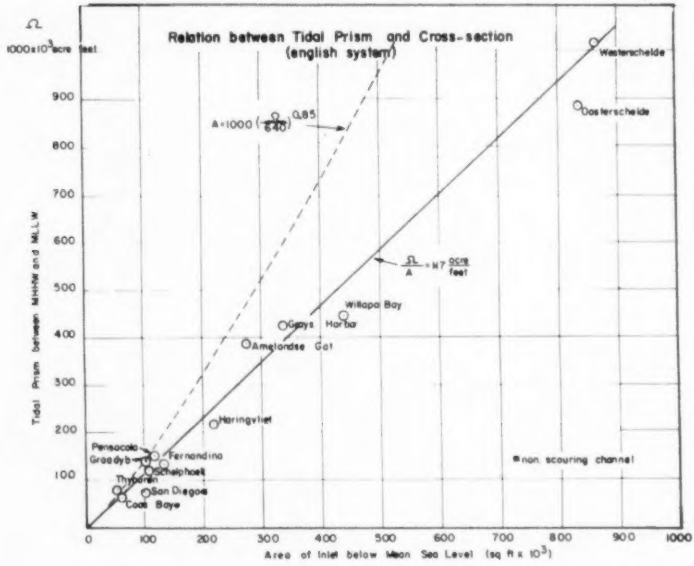


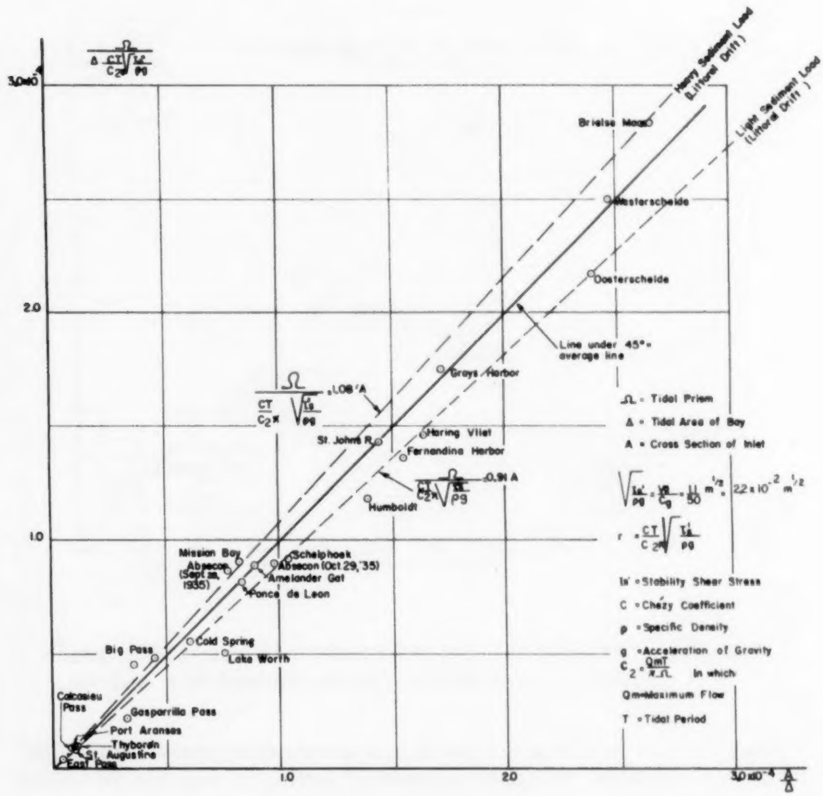
Fig. 5b: Statistical relationship between tidal prism and cross section for small and big inlets.

Two lines have been drawn in the figure, one on each side of the line for average t_s' indicating the relation between Ω and A, for high and low values of t_s. The differences from the average, respectively +8 and -9%, are proportional to the square root of the stability shear stresses for the different inlet. That means that with respect to the average, the shear stresses vary $\pm$ 17% under the simpler conditions upon which the study is based.

The combination of phenomena as discussed earlier are responsible for the deviations. It is not yet possible to determine the influence of the different factors involved separately. For inlets of more complex configuration the shape and configuration factor must be expected to be rather important. In Fig. 7, the value $V_m = C\sqrt{\frac{\tau_m}{\rho g}}$ is plotted against the ratio $\frac{\Omega}{AT}$ for the inlets under consideration. Most of the inlets have maximum velocities between 3.1 and 3.9 ft./sec. Most likely the littoral drift is an important agent indirectly or directly. A part of the difference may also be caused by differences in bottom material (grain size) and exposure to wave action.

Man's Efforts in Improving the Stability of Coastal Inlets

The development of navigation made it necessary to provide harbors for an increasing number of vessels of increasing size. It became necessary to



Fig, 6: Relationship between tidal prism and cross section in a dimensionless form.

fix the position of inlets and estuaries, dredge wider and deeper navigation channels and protect these channels against unpredictable wave and current action and against shoaling.(28)

It is outside the scope of this paper to discuss technical details of such regulatory steps but referring to the preceding paragraphs, it is self-explanatory that definite knowledge of the relationship between flow and inlet geometry is of vast importance as is a thorough understanding of other pertinent physical factors involved. Methods for inlet improvements are dredging operations, protection of the inlet against littoral drift deposits by jetties, and contraction of the inlet channel by dams, groins and spurs. By-passing sand plants with the double effect of limiting the erosion on the downdrift side of the inlet and decreasing the amount of deposits in the inlet channel have been built in recent years. As mentioned above, theories have also been developed for determination of the ideal shape of an inlet estuary. But in the

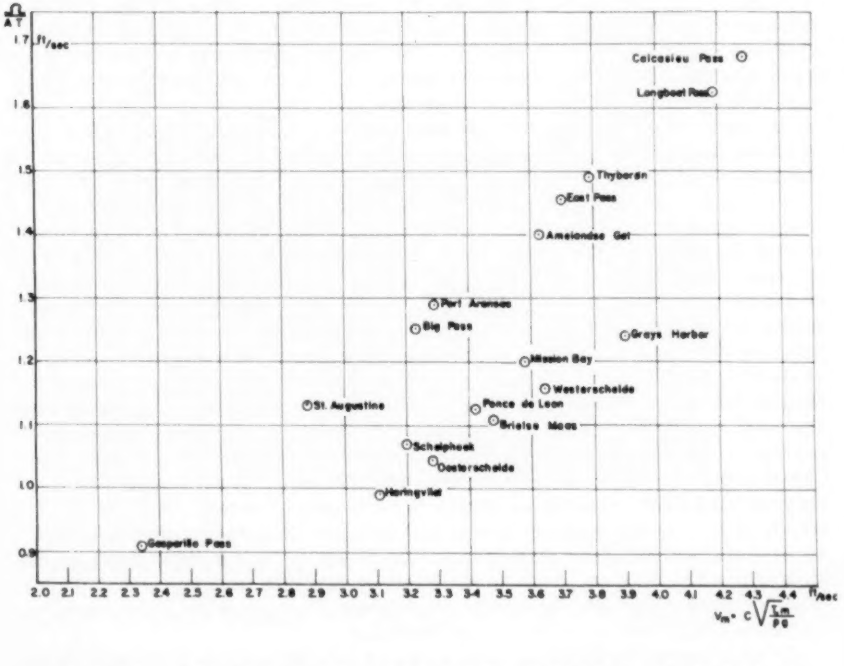


Fig. 7: Maximum velocities for different inlets.

overall picture, the result is still that no integrated method exists which can be applied with confidence to the solution of tidal inlet problems. Recently a preliminary report was printed by the Beach Erosion Board.(1) The results are interesting from a qualitative point of view, but do not allow any conclusion of quantitative nature. The coastal engineer will enjoy seeing some of his experience reproduced in a scale model, but will at the same time regret that little was gained useful for design.

This question can therefore be raised: are we on the right track with our efforts? Certainly we are. With a limited amount of funds, much knowledge and experience has already been gained. We must realize that we are coping with a problem which involves ten to twenty variables if not more. These variables can be combined in more than a hundred different ways and even if most of these combinations are considered unrealistic, we still have so many combinations remaining that it is all but impossible to give a definite answer to any particular problem based on experience only.

What we can accomplish is that for a given "desirable cross-section" we can calculate the size of the tidal flow necessary to keep this cross-section fairly stable, and vice versa, taking the actual situation of tides, bottom material, silt-load and different friction elements into consideration. As described above, we already have a certain amount of data available for such purpose.

A choice can be made for the value of the stability shear stress, taking the different inlet characteristics into consideration, in which Lane's data (Table 2, page 30) are very helpful. The friction factor can be calculated directly or on its relationship to Manning's n. The coefficient C_2 is a function of Keulegan's coefficient of repletion.

In other words we are probably able to give the necessary data for a preliminary estimated design, but such design would be based on average conditions, and nature does not always respect the statistical mean. Heavy storms pour littoral drift deposits in the inlet channel regardless of how "ideal" its cross-section is, compared to the "average flow." The technical answer is jetties as protection again "surprises." Jetties will result in a different or special type of cross-section development, but we are still able to handle this "forced cross-section" by our tidal computations. The shape factor of the cross-section is now out of consideration because the shape is given as approximately rectangular, the influence of littoral drift is incorporated in the choice of t_s and the calculations of flow conditions is much simplified. Yet such simplified case is in fact not yet adequately considered. The result of such consideration is most likely a more economical and improved relationship between Ω and A. Reference is made to the below mentioned above the actual influence jetty improvements. If we install a by-passing sand plant, everything seems to be in proper order. Yet the experienced coastal engineer knows that what we have accomplished is that we have moved one more step in the right direction still without being saved. Most likely the inlet will still be bothered by deposits in the channel. These deposits often come very suddenly, restricting the size of the cross-section. It will be of interest to consider the following.

At the Coastal Engineering Laboratory of the University of Florida, model experiments with fixed bed have been made to study the influence of a deepening of Lake Worth Inlet, Florida (to 35 feet below M.L.W.) on the behavior of the watertable in the bay (Lake Worth) in order to accommodate vessels of greater draft to enter the Port of Palm Beach. The Lake Worth Inlet is protected by two parallel jetties, perpendicular to the shoreline.

It is outside the scope of this paper to discuss this study, but the flow conditions in the inlet channel have a considerable interest for this paper. The flow at the entrance is, during flood, subject to contraction, as shown in Fig. 8a of the improved inlet. In nature littoral drift deposits have accumulated on the updrift side of the northern jetty. South of the inlet erosion has taken place resulting in a greater effective length of the southern jetty. Because of this difference in "actual length" between the two jetties, the flow pattern is asymmetrical.

Fig. 8b depicts the ebb-flow pattern which shows agreement with the jet-flow theory because no natural tidal channels outside the inlet are present. Reference is made to French's progress reports,[18,19] dealing with the velocity distribution in tidal entrances.

In order to investigate the influence of the contracted flood-flow in the case of parallel jetties for a group of inlets the cross-sections at different locations in the inlet have been plotted against the distance from the seaward end. The diagram (Fig. 9) has been made dimensionless by plotting A/A_0 against x/L, in which A_0 is the cross-section at the end of the jetties and L is the length of the inlet. Except in the case of Fernandina Harbor, the entrance cross-sections are greater than in the rest of the channel. Material is deposited in the inlet channel by the concentrated flood currents and the eb'

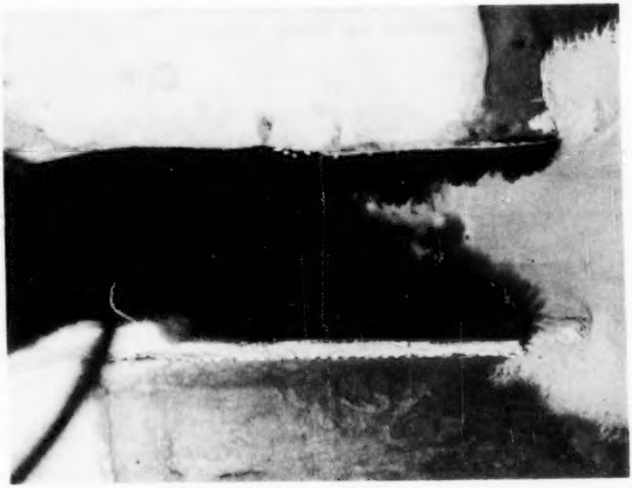


Fig. 8a: Flood flow in Lake Worth Inlet according to model experiments at the Coastal Engineering Laboratory of the University of Florida.

Fig. 8b: Ebb flow in Lake Worth Inlet according to model experiments at the Coastal Engineering Laboratory of the University of Florida.

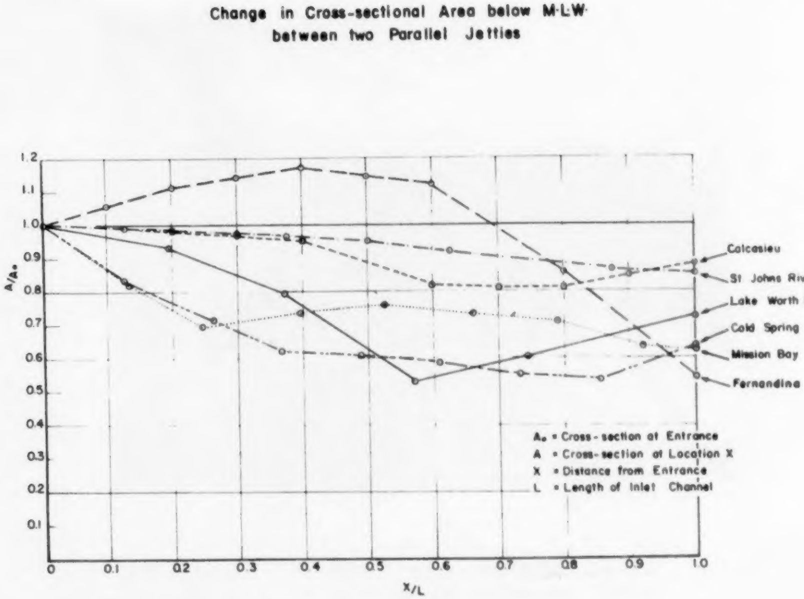


Fig. 9: The cross-section between two parallel jetties as a function of its location in the inlet channel.

currents are not strong enough to clean the channel satisfactorily.

Another example of an improved inlet is Grays Harbor, Washington, on the Pacific Coast (Fig. 10a and 10b). To protect the entrance from shoaling (predominant littoral drift is to the south) a south jetty was built (1898 to 1902) and a north jetty (1907 to 1916). These are convergent in the direction of the entrance, and have improved the situation considerably with respect to navigation. Although a new bar formation has been formed in front of the new entrance (outside the jetties) a stable navigation channel has been obtained by great amounts of annual dredging of the bar. No dredging in the outer bar has been required since 1942. Unfortunately the improvement of the harbor has created a serious erosion problem at Point Chehalis, which problem has not yet been solved.

In the case of the Thyborøn Inlet in Denmark a great number of groins were built as inlet protection. Dredging work is the outer bar was unsatisfactory. When major dredging operations were undertaken on the bay shoals the situation improved and a jetty constructed out over the outer bar on the updrift side of the inlet improved the situation furthermore and materially. The problem of keeping free of too much trouble with sand deposits apparently can be solved, partly or wholly, in two different ways:

1. By dredging a so-called "non-scouring" channel, (tidal prism too small for the cross-sectional area) so that only low current velocity occurs in the

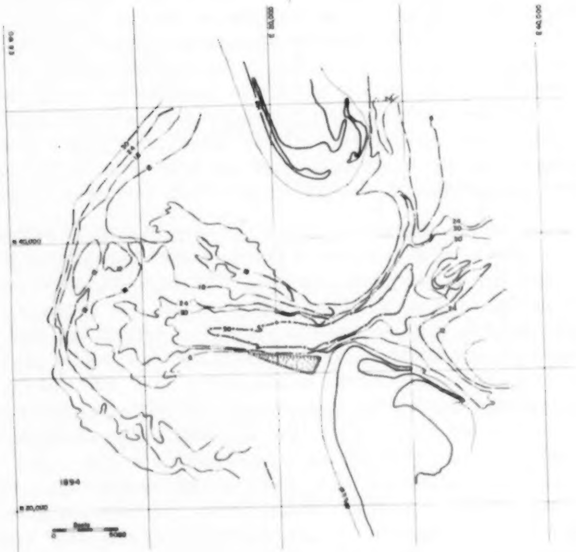


Fig. 10a: Grays Harbor Before Improvement (1894)

Fig. 10b: Grays Harbor After Improvement (1954)

inlet channel. Part of the cross-section in this way functions as a "sand trap" for littoral drift deposits. An example of this is San Diego harbor in California. This kind of inlet does not give rise to the big surprises. The non-scouring channel must be characterized as a good—but also expensive-method. However, high initial costs may be justified in more stability and in less maintenance. But a non-scouring channel cannot be used just anywhere. Special conditions such as a comparatively small tidal prism must be available. Needless to say the more important the inlet from an economic point of view, the more justified the development of a "non-scouring" channel.

2. By model studies. Numerous model studies of harbor and inlet problems have been made. The purpose of these studies has always been to examine or check the design more closely. The importance of such studies is to assure the highest possible benefit of the tidal flow through the inlet by giving the inlet and bay channel and the protective jetties the best configuration and cross-sections for the purpose. Such studies can be carried out with fixed as well as movable beds. In the latter case, littoral drift experiments can be included so that the influence of the jetties on the longshore drift can be investigated at the same time and the jetties designed accordingly.

A considerable amount of experience is available and in some cases model experiments function primarily as a check on the design. Meanwhile the experienced coastal engineer knows that one can never take out insurance enough against surprises from behavior of tidal inlets on littoral drift coasts—and they will for this reason often prefer to let survey and model studies provide a maximum amount of information before construction starts—instead of trying to tell nature how it is expected to behave according to their experience. The cost of such experiments are usually so low that there is little or no economic justification for omitting them. The time factor involved in a model study should not be of any great importance when high costs are involved.

Examples of such studies are not so numerous in the United States where less opportunity has been given for such experiments as elsewhere. The Absecon Inlet study (Corps of Engineers, Philadelphia District) provided a number of interesting results. The best known model experiments of this nature have been the Dutch experiments with the harbors of Abidjan, Lagos and Ada on the west coast of Africa. Similar experiments are presently in progress in England, Holland, France, India, Denmark and at several other places in the world. It is believed that any cost involved in these studies will eventually pay satisfactory dividends.

CONCLUSION

1. A considerable amount of experience is available in regard to the design of tidal inlets. However, more information about pertinent elements of hydraulic and physical nature is desirable in order to furnish more detailed design data. Such information must include evaluation of the shape factor of the inlet and of factors governing the determining tractive force as suspended and bed load materials. Also the influence of fresh water outflow should be evaluated more closely.

2. With our present state of knowledge about tidal inlets on littoral drift coast, it is considered desirable also to evaluate the detailed design by model experiments securing the best utilization of the tidal flow. Progress in the

tidal inlet hydraulics field will permit more and better definite conclusions based on calculations and experience, but it is considered being desirable anyhow to check the design by model experiments in order to secure a maximum of knowledge and experience as basis for design data.

ACKNOWLEDGMENT

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CONTENTS

DISCUSSION
(Proc. Paper 1653)

THE LITTORAL DRIFT PROBLEM AT SHORELINE HARBORS[a]

Closure by J. W. Johnson

J. W. JOHNSON,[1] M. ASCE.—As stated in this paper, wave action is the primary source of energy available at a shoreline for moving sediment. Waves may be generated either by local winds or by storms at a relatively great distance from the shoreline under study. Locally generated waves generally are referred to as "seas" and waves generated by a distant storm are referred to as "swell." The reader unfamiliar with the basic elements of wave generation and wave theory is referred to the publication by Bigelow and Edmonson.(1) Workable relationships between the wave characteristics and a generating area are well established, and graphical methods are available for estimating these wave characteristics (height, period, and direction) from known wind conditions. Most useful to the practicing engineer are the procedures outlined in recent reports.(2,3,4) By the use of these principles "hindcasts" of wave characteristics have been made for many coastal regions —particularly in the United States. For example, wave statistics are available for the Pacific Coast from Cape Blanco, Oregon to the California-Mexico border,(5) The Gulf of Mexico(6-11) the Great Lakes,(12-14) and the North Atlantic seaboard.(15-17) Less accurate in detail, but of considerable value for preliminary analyses, are the frequency of wave conditions in the principal ocean areas of the world as presented by Bigelow and Edmonson.(1) These latter summaries present the distribution of high seas and high swells for the principal ocean areas of the world for various months of the year. The character of the winds which produce these wave conditions also are discussed.

An example of wind and wave conditions at a given locality is presented in Fig. 9 which was prepared in 1912 by Spring.(17) This sketch shows the reversal of drift at Madras Harbor to be the result of monsoons which approach the coastline from S.W. at one season of the year and the N.E. at another season. More recently Manohar(18) has discussed the wind systems on the Indian Coast and their effect on the littoral drift at Madras and other South Indian Ports. In general, only a careful study of meteorological conditions for a particular location will yield information on the wave conditions which determine the movement of littoral drift. An example of such a study is that conducted by Munk and Traylor(19) on the typical meteorological situations occurring in the widely separated areas of the Pacific Ocean which give rise to the characteristics of waves on the Pacific Coast of the United States with special reference to the waves at La Jolla, California (Fig. 1). Along the Southern California coast the predominant drift is in a southern direction and

a. Proc. Paper 1211, April, 1957, by J. W. Johnson.

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is caused by both seas and swell from the Northwest. The reversal observed during the summer months is the result of the swell from the Southern Hemisphere. A critical analysis of the above discussion and references on wave statistics will show the fallacy of Mr. Silvester's statement that it is "The rule rather than the exception that storm waves approach a coast from a different direction from the predominant swell." Such a generalization certainly is misleading to the novice in the field.

Mr. Silvester questions the figure 0.025 as the value of the critical steepness at which the character of the bottom changes occur near shore; however, he neglects to state whether he believes that the value is too low, too high, or a variable. Examination of the observations on wave conditions and beach changes by Wiegel, Kimberly, and Patrick(20) on a natural beach would indicate that the value of 0.025 obtained in laboratory studies may be higher than that which occurs in nature; however, additional field data are necessary to more accurately establish this critical value.

Nearshore sediment movement, which has been described elsewhere,(21) consists of movement by suspension and rolling in the turbulent region of the surf zone and possibly by turbulent forces at fairly great depths, seaward of the shoreline. Apparently these latter turbulent forces from wave action are great enough to cause a general alongshore movement of sand out to depths as great as 170 ft.(22) Although oceanic currents in some localities may be of sufficient strength to transport sand along the sea bottom, the primary mechanism of sediment transport in deep water appears to be the result of turbulence resulting from the oscillatory motion at the sea bed by surface waves. Sediment placed into temporary suspension by such turbulence then may be transported along the sea bottom, and possibly around headlands, as a result of either or both of the following forces: (a) an oceanic current, or (b) when the forward velocity of the water under the wave crests exceeds the backward velocity under the trough, a net transport occurs in the direction of the wave travel.(23) Four rather comprehensive investigations of the fundamental mechanics of sand movement by oscillatory waves have been made recently by Li,(24) Manohar,(25) Kalkanis,(26) and Vincent.(27) A summary of the results of these studies is beyond the scope of this discussion; however, it should be appreciated that because of the action of surface waves, there are initial and general movements of sediment, as well as the initiation, various stages of development, and complete disappearance of bed undulations or ripples. The movement of sediment takes place in a boundary layer that is developed at the sea bottom from the effects of the viscosity of the fluid. The initial and general motion of small sizes of sediment is caused by laminar shear, and similar motions of large sizes of sediment are caused by lift forces in a turbulent boundary layer. Ripples, in general, are not formed unless the flow is turbulent in the boundary layer. All motion in turbulent flow and the various stages of development of ripples have been found to be functions of a dimensionless function representing intensity of flow of the fluid near the bottom.(25)

Mr. Silvester states that the motion on the sea bottom beyond the surf zone is by "saltation." If the term saltation is used in the same sense that it has been used in sediment transportation in streams since the classic work of Gilbert(28) in 1914, then Mr. Silvester's belief that this factor is "the most important factor in littoral drift processes" is in error. Kalinske's analysis of a criteria for determining sand-transport by surface creep and saltation(29) conclusively demonstrated that movement by saltation in flowing water is of

minor importance; that is, "The height of grain-rise in the saltation-phenomena in water will be of the order of 1/1000 of that in air, or for most practical conditions, a few grain-diameters." The author therefore suggests that the use of the term "saltation" in connection with littoral drift problems be avoided—as was done purposely by the author in the original paper.

The movement of sand from offshore areas depends on the hydrography and predominant wave conditions. Attempts to nourish an eroded shoreline by dumping material a short distance offshore by hopper dredge, with the assumption that the material would be moved onshore by wave action, has met with only partial success.[30] For example, in such an operation at Long Branch, New Jersey, it was concluded that over a 4-year period there was no evidence that material moved ashore from the stockpile or that the shore was benefited by the operation.[31]

In a discussion of the general problem of offshore sediment movement Mr. Silvester cites studies at Portland Harbour, Australia on determining the path of sediment movement by tracer minerals. The report on this study shows that the sediments appear to follow smooth paths somewhat identical to the orthogonals to the wave fronts. There is no evidence in this presentation, however, that a sufficient frequency and number of samples were taken to establish that the material moved around the harbor area in the manner indicated.

Mr. Silvester had difficulty in envisaging "the necessity of bypassing material in both directions at different times of the year." The author has no difficulty in envisaging such an operation, but does believe that there are less expensive methods of keeping a relatively narrow harbor entrance open where important reversals in littoral drift occurs. An excellent example of where the reversal in direction of bypassing sand might be practiced is the Camp Pendleton harbor shown in Fig. 24.* The rate of drift, reversal of drift, width of opening, and tidal prism are such that navigable depths in the entrance channel cannot be maintained without continuous bypassing (in one direction or the other) or by continuous dredging in the channel itself. What appears to be a less expensive procedure in maintaining this harbor in an operational condition is to realign the jetties as proposed by the Corps of Engineers, and shown in Fig. 2. "Principal features of this plan would be to modify the jetties and develop a sand-bypassing operation that would be adequate to maintain navigable depths in the entrance to the harbor. The purpose of the jetty modification would be to cause deposit of littoral drift in a protected area behind the extended north jetty from which it could be readily removed by floating plant and deposited along the shore at Oceanside."[32] The predominant southerly drift is expected to accumulate as shown in Fig. 2. The northerly drift resulting from the southern swell (Fig. 1) will also deposit in a protected area where it also can be removed periodically by a floating dredge.

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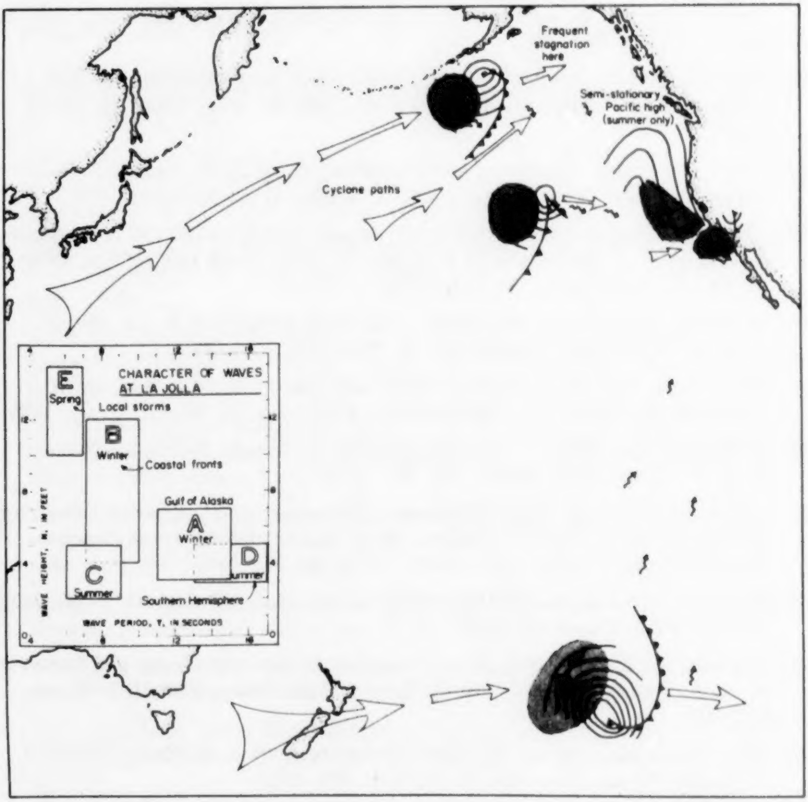

Fig. 1 — Origin of Waves reaching La Jolla, California. Capital letters on map represent typical meteorological situations which give rise to the character of the waves shown in the insert. Shaded areas represent typical fetches or generating areas. Isobaric patterns and nature of fronts also are indicated. Large open arrows represent typical paths of the storm systems, and the wavy arrows indicate the waves leaving the storm system.

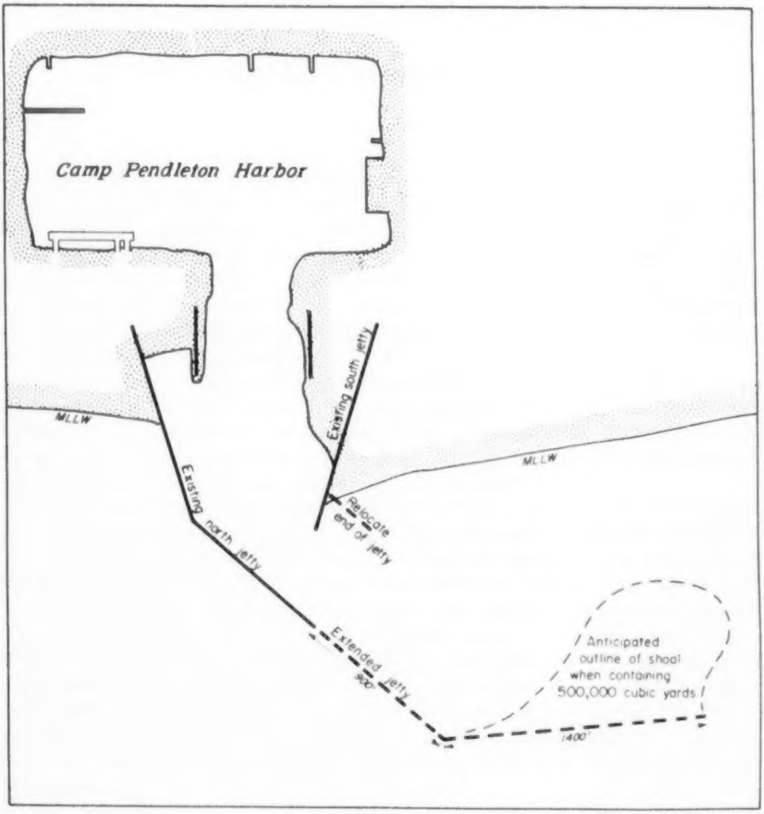

Fig. 2 — Proposed changes to Camp Pendleton Harbor jetties to provide a protected stockpile area for sand bypassing.

THE COLUMBIA RIVER CONTROLLED[a]

Discussion by Roy F. Bessey

ROY F. BESSEY,[1] M. ASCE.—General Foote's broad and informative paper is very timely, as will be the Corps' of Engineers' forthcoming report on the Columbia River and tributaries which the paper previews.

The 1948 review report of the Corps represented a considerable step forward from that of 1932 in concepts of comprehensive, multiple-purpose development of a major river basin. In the view of this writer, the 1958 review report should present another forward step, with new emphasis on "full" development and use of resources.

A plan for full development faces a number of evident hazards in the form of opposition to various elements by and controversy among various interests affected, as well as in various proposals for piecemeal or limited-purpose development. These opposing forces can cut very deeply into the heart of a comprehensive, rational, and widely beneficial plan.

Yet the need of full development of basic river basin resources is greater than ever. In the face of rapidly expanding population, of rising living levels, and of the growing material, economic and social needs of a strong, dynamic and highly productive economy, we cannot do other than strengthen and widen our resources base, conserving and using our natural resources to the full. This is reason enough for a full development plan. But such a conclusion should be inevitable in view of a continuing, long-run world crisis in which our strength and leadership in the scientific development and use of resources is a matter of great moment both at home and abroad. It is underscored by a wakening knowledge of strong advances in the technology and development of vast river basins, and other resources, by our principal rivals in world power and influence.

It will be evident, as General Foote implies, that problems of planning and development call for more than the best available engineering science. The problems of economics, of negotiation, of legislation, of organization and co-operation—of political science particularly—should be apparent also.

The case for the full development of the Columbia River system is especially strong. This is the Nation's greatest power stream. The Columbia ranks among our greatest in water supply potential. It has outstanding navigation potentials, particularly so far as the West is concerned. The resources of the Columbia are thus of truly large national consequence, and their full development and contribution toward meeting the needs of the national economy and of our defense and security are distinctly in the general interest.

A need of breadth of view as to comprehensive, multiple-purpose, and

a. Proc. Paper 1514, January, 1958, by Louis H. Foote.

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optimum development of the Columbia is implicit in General Foote's paper, and was stressed by him in recent public hearings on the review report in process. It is a main purpose of this discussion on the part of the writer to support such a view, as well as to stress further the politico-economic side of the problem of such development.

This writer believes that the views outlined below, a condensation of his testimony as presented to the Corps of Engineers in the public hearings referred to, are pertinent to professional engineering—as well as official and political—consideration of the tentative plans and alternatives so ably sketched by General Foote.

Public Interest, Public Need, Basic Principles and Public Policy

The national and public interests involved demand the rigorous application of well established principles of conservation and development, and the firm establishment of the best-feasible plan and program of development.

The public interest in national resources such as are here involved is paramount. The wise use of such resources is linked inseparably with underlying purposes of human enterprise, security and well-being. Development of those resources should make maximum contribution to fulfillment of the great and growing material needs of the Nation, region and localities. They should make maximum contribution to national, and free world, security. They should make their vital contributions to pressing regional and local needs for expanded and diversified economic opportunity, both for purposes of recession-relieving and employment-stabilizing activity through construction and related industry in the immediate situation and of strengthening industrial and general economic base in the longer run.

The hard-won principles of wise and full conservation and use of resources, of comprehensive and coordinated, multiple-purpose, basin-wide development, of maximum net benefit and wide distribution of benefits, should be applied to the feasible limit.

A comprehensive plan, and a clear program for Federal development, should be recommended by the Corps to provide, with maximum responsibility, effectiveness and economy, the highest feasible objectives and levels of ultimate development. Also an immediate phase should be laid out to meet fully the estimated needs of the next two decades or more.

Coordinated Main Control Plan

A coordinated main control plan providing maximum feasible storage and regulation of flow is required as a key to the achievement of maximum benefits, well distributed among multiple uses and geographic areas. The high sights of the 1948 plan, in its current and ensuing phases, should not be lowered. The goals of the plan for the next two decades or less should include storage capacity, well located and distributed, to reduce the flow of an 1894-volume flood to about half or to not over 600,000 c.f.s. at The Dalles, to provide maximum at-site and downstream power, to insure adequate water supplies in quantity and quality for rapidly expanding industrial, agricultural, municipal and domestic uses, and to provide other material benefits.

The gross and net benefits of strategically-located, large-capacity, upriver storages of a main control plan—such as the large Hells Canyon, Nez

Perce, Paradise, Libby, and Canadian projects—are greatly superior in scope and economy to the limited-purpose and limited-scope projects that would bar development of the larger projects and result in permanent losses of resources, project and program capabilities and benefits. Such key projects are essential to attainment of optimum river regulation and should be included in the ultimate and immediate programs.

Individual and Combined Aspects of Development

Some such aspects of development aims and criteria should be especially stressed:

Water Storage

The retention of excess flows and release at more favorable times—is, of course, a fundamental purpose and procedure in river system development and utilization. The uses of storage in flood control, in power production, and in water supplies for various purposes are readily apparent. On the Columbia, where floods are primarily of the more predictable snow-melt type, the conflicts among storage uses are relatively minor; the uses for flood control and for power, for instance, are quite complementary and mutually supporting. The main burden of establishing feasibility of storage development does not rest either upon flood control or upon power, but upon both. Adequate storage capacity, in short, is a key to comprehensiveness and effectiveness in a main control plan with a balanced upstream and downstream development in which feasibility of each of those sides of development is enhanced by the other. Under the circumstances, it is desirable to aim at the highest feasible level of storage development.

Power

The vital and rapidly expanding needs of the national economy and the region for energy exert great pressure for the most complete and economical development of the unparalleled water-power resources of the Pacific Northwest. The importance of abundant electrical energy at lowest practicable cost in our progress and our security can hardly be overstated, nor can that of use of the Columbia's full potentials. Obviously, that full potential in maximum output and low unit cost cannot be attained if the at-site and downstream values of upstream projects go unrealized.

Under the anticipated load growth of the next two decades or so, power requirements will pass, as General Foote, shows, beyond the combined power capacity of all feasible hydro projects. Since the unit costs of such hydro power will be less than that of thermal power from any source, all feasible hydro should be developed; not to do this would be stark waste of a continuing, self-replenishing resource.

The lower unit cost of power from the large Federal projects—due to economies of large scale, of multiple-purpose development, of hydraulic and electric integration, and of public financing—should also be considered for its spiralling and cumulative effects: the lower cost will induce higher usage and economic activity, raising rate of load growth, and increasing power benefits. The low energy cost is crucial in the location and establishment of certain heavy industrial plants with their primary effects in economic expansion and diversification, badly needed in the region.

For such reasons it is suggested that load estimates for power development can be too conservative—tending to inhibit full and timely development. The point is that one way to avoid power "shortage" is to hold growth down—to fail to meet the need.

Although power benefits do not stand alone and must be considered in conjunction with other multiple benefits of a coordinated plan, they are largest among individual benefits in monetary and economic terms. The economic returns from high-volume-low-cost power—in added industrial development and diversity, in raised production, income and living levels, and in increased strength, balance and security in local, regional and national economies—are very great and broadly proportionate with the volume and economy of the power made available.

Flood Control—with dependence primarily upon upstream storage (on Columbia and up-river tributaries and on the Willamette) and secondarily upon local levee systems—is essential for protection of life, property and productivity, and for effective use of strategic and highly-valuable lowland areas for their best economic purposes. Full weight must be given, of course, to the direct and indirect, measurable and unmeasurable, human values of flood control—to the savings in human life and welfare, in individual and family well being, and the prevention of losses and dislocations in industry, service and community activities, and in production and income.

Again, maximum feasible main control plan goals should be set, as indicated above. As the Division Engineer said in his preliminary bulletin on the current study, "a greater degree of control by storage is desirable and is obtainable within the limits of economic and needed development of the water resources for the generation of power."

Navigation development of the West's outstanding seaway and inland waterway reaching far into the interior—with maximum feasible geographic extension and channel and lock improvement to highest practicable standards—is essential to realization of economic potentials of the river system and the region, and will provide a material contribution to the national economy.

Water supply enhancement and security, in one of the Nation's top-ranking resources of the kind—namely the Columbia—is necessary to meet widely-recognized and growing national and regional needs, particularly for crucial demands of industry, but also for vital agricultural, municipal and domestic requirements.

Combined uses—particularly of the strategic combination of navigation, abundant and low-cost power, and adequate water supplies, coupled with local port, terminal and industrial district improvements—should be given full and coordinate consideration in a basic plan and program. Flood control combines with these strategic elements in enhancing the developmental values inherent in the waters and waterfronts of main-stem areas.

Irrigation that can be provided through the development of the Columbia is of primary importance. It is essential that the maximum feasible acreage be developed as of value both in areal and regional economies and in meeting the expanding food and fiber needs of a growing West and Nation. The new crop land is needed, in spite of increasing efficiency of production, in order to meet requirements of an expanding population coupled with substantial withdrawals of crop land areas for various special purposes (including those of urban, industrial, transport, and military usage) as well as for reasons of agricultural submarginality.

Fisheries are of very large consequence not only because of the positive

economic and social values in the Columbia system, but because of the deterrent effect of fishery problems and controversies upon a full and timely multiple-purpose development of the rivers. It is highly desirable that the fishery aspect be considered from a progressive and developmental, and not merely a protective, standpoint. An entirely adequate and fully integrated attack on the fishery part of the development problem has not been heretofore attained; essential fishery research, planning and improvement work has tended to lag behind in the procession of river project development. There is need of a new initiative in picking up neglected opportunities for extending the fisheries' capabilities both as to area and volume. The solution of the problem of passage at high dams is particularly pressing from the standpoint of fish resource protection, but it may also open—or even reopen—long river reaches for fish production. Fishery research, planning, and experimental work should be stepped up to proceed in advance of prospective river changes, and fishery improvement should proceed in close concert with river development works if maximum values are to be obtained in fishery and general development.

Other beneficial water uses—including watershed management, recreation, and wildlife, mentioned by General Foote, are not here discounted in any sense. They are of great and growing economic and social significance and should be considered as primary objects and benefits in both immediate and long-range plans and programs of development.

Project area planning is a field in which a larger and better coordinated effort would pay extra dividends in project and program benefits. A major river development project will be of greatest value and will have greatest assurance of successful consummation if its integral nature and close relationship to its environment is preserved—if all of the mutually supporting uses are developed to the full, included in plan and authorization, and carried out in coordination.

In this field responsibility and leadership must be shared among Federal, state and local agencies. Coordinated project, land, community and utility planning should proceed, with project planning and development, with the view of minimizing inevitable dislocations and reaping maximum benefits in timely and orderly re-establishment and establishment of farms, industries, homes, communities, transportation, electric utilities, governmental services, and other facilities and activities affected.

In this connection, preliminary draft legislation for the large Paradise project would make interesting and innovating provisions for area replanning under Federal sponsorship. Past joint investigations and planning activities headed by the Bureau of Reclamation in connection with its Columbia Basin project provide a useful precedent.

Coordination of effort. The difficulties in putting together and holding together a complex plan and program are great—a condition that is widely recognized as a result of long and varied experience in the Columbia and other basins. Accordingly, it is suggested that special attention should be given to problems and needs of organization and procedure and other ways and means for coordination of effort in planning and programming, public information and cooperation, legislation and authorization, appropriation and finance, and construction and operation aspects and stages of development.

Federal Aims and Responsibilities

A Federal purpose and responsibility for the wise conservation, development, and management of national resources, for most effective improvement of major river systems with maximum aims in beneficial use for all purposes, shows clearly in the constitutional, legislative and policy background of river basin investigations and planning, such as that presented by General Foote. In the present crucial situation, with its clear need for expansion in resource base and with a momentous review of the existing comprehensive plan for the Columbia system coming into focus, the Corps of Engineers clearly holds a responsibility for drawing the best—the most comprehensive and most effective—development plan possible.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

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MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238(WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW2), 1245(HW2), 1246(HW2), 1247(HW2), 1248 (WW2), 1249(HW2), 1250(HW2), 1251(WW2), 1252(WW2), 1253(IR1), 1254(ST3), 1255(ST3), 1256(HW2), 1257 (IR1)[c], 1258(HW2)[c], 1259(ST3)[c].

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267(PO3), 1268 (PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275(SA3), 1276(SA3), 1277 (HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283(HY3)[c], 1284(PO3), 1285(PO3), 1286 (PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296(HW3), 1297 (HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303(ST4), 1304(ST4), 1305(SU1), 1306 (SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312(ST4), 1313(ST4), 1314(ST4), 1315 (ST4), 1316(ST4), 1317(ST4), 1318(ST4), 1319(SM3)[c], 1320(ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324 (AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328(AT1)[c], 1329(ST4)[c].

AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337(SA4), 1338 (SA4), 1339(CO1), 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345(HY4), 1346(PO4)[c], 1347 (BD1), 1348(HY4)[c], 1349(SA4)[c] 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360 (IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369 (WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374(HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c], 1378 (HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4), 1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395 (HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404 (HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413 (EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417(HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

NOVEMBER: 1426(SM4), 1427(SM4), 1428(SM4), 1429(SM4), 1430(SM4)[c], 1431(ST6), 1432(ST6), 1433(ST6), 1434(ST6), 1435(ST6), 1436(ST6), 1437(ST6), 1438(SM4), 1439(SM4), 1440(ST6), 1441(ST6), 1442(ST6)[c], 1443(SU2), 1444(SU2), 1445(SU2), 1446(SU2), 1447(SU2), 1448(SU2)[c].

DECEMBER: 1449(HY6), 1450(HY6), 1451(HY6), 1452(HY6), 1453(HY6), 1454(HY6), 1455(HY6), 1456(HY6)[c], 1457(PO6), 1458(PO6), 1459(PO6), 1460(PO6)[c], 1461(SA6), 1462(SA6), 1463(SA6), 1464(SA6), 1465(SA6), 1466(SA6)[c], 1467(AT2), 1468(AT2), 1469(AT2), 1470(AT2), 1471(AT2), 1472(AT2), 1473(AT2), 1474(AT2), 1475(AT2), 1476(AT2), 1477(AT2), 1478(AT2), 1479(AT2), 1480(AT2), 1481(AT2), 1482(AT2), 1483(AT2), 1484(AT2), 1485(AT2)[c], 1486(BD2), 1487(BD2), 1488(PO6), 1489(PO6), 1490(BD2), 1491(BD2), 1492(HY6), 1493(BD2).

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JANUARY: 1494(EM1), 1495(EM1), 1496(EM1), 1497(IR1), 1498(IR1), 1499(IR1), 1500(IR1), 1501(IR1), 1502 (IR1), 1503(IR1), 1504(IR1), 1505(IR1), 1506(IR1), 1507(IR1), 1508(ST1), 1509(ST1), 1510(ST1), 1511(ST1), 1512(ST1), 1513(WW1), 1514(WW1), 1515(WW1), 1516(WW1), 1517(WW1), 1518(WW1), 1519(ST1), 1520 (EM1)[c], 1521(IR1)[c], 1522(ST1)[c], 1523(WW1)[c], 1524(HW1), 1525(HW1), 1526(HW1)[c], 1527(HW1).

FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c].

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c], 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616 (HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

c. Discussion of several papers, grouped by divisions.